Formal SPE Submittal, Geotechnical Report

Building X
Redmond, Washington

for
Willow Run, LLC

October 16, 2019

GeoEngineers
Earth Science + Technology

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File No. 23237-002-01

October 16, 2019

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INTRODUCTION

This report presents the results of GeoEngineers, Inc.'s (GeoEngineers) preliminary geotechnical engineering services for development of the site located at 10301 Willows Road NE in Redmond, Washington. The site is approximately trapezoidal in shape and is bounded by adjacent properties to the north, south and west, and Willows Road NE to the east. The site is shown relative to surrounding physical features in Figure 1, Vicinity Map and Figure 2, Site Plan.

The purpose of this report is to provide geotechnical engineering conclusions and recommendations for the design and construction of the planned development. The site consists of one King County parcel (parcel number 342605-9037) and covers approximately 8.9 acres. GeoEngineers' geotechnical engineering services have been completed in general accordance with our professional services agreement executed on July 31, 2018. Our scope of services includes:

- Reviewing existing subsurface information available for the site and surrounding area;
- Completing explorations at the site to further characterize subsurface and groundwater conditions;
- Completing geophysical testing for shear wave velocity measurements and vibration monitoring;
- Completing slug testing to evaluate hydraulic conductivity of onsite soils;
- Providing a site-specific response analysis;
- Providing recommendations for earthwork;
- Providing foundation, temporary shoring, slab-on-grade and permanent below-grade wall recommendations;
- Providing recommendations for temporary and permanent dewatering and groundwater seepage estimates; and
- Preparing this report.

PROJECT DESCRIPTION

GeoEngineers understands that the project team is interested in developing a multi-story office building and parking garage at the project site. Excavation depths for the planned development are anticipated to range up to 25 feet below existing site grades.

Temporary shoring is anticipated to complete portions of the excavation for the planned building. Localized temporary dewatering will also be required. Foundation drainage is planned to allow the building to be designed using conventional below-grade walls and slabs-on-grade.

Competent soil conditions are present at the anticipated foundation elevation in the western portion of the site. Where competent soils are present at the foundation elevation, spread or mat foundations are feasible. In the eastern portion of the site, fill and recent deposits are present. The fill and recent deposits are moderately compressible under static loads and potentially liquefiable under seismic loading. Where fill and recent deposits are present at the foundation elevation, deep foundations or ground improvement will be required for foundation support.

FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

The subsurface conditions at the site were evaluated by drilling 17 borings, GEI-1-18 through GEI-12, and GEI-19 through GEI-23, to depths ranging from approximately 26 to 101½ feet below existing site grades. GEI-13 through GEI-18 were completed on a separate property to the south of the project site. The approximate locations of the explorations are shown in Figure 2. Five of the borings were completed as monitoring wells and boring GEI-19 was completed as a 6-inch diameter test well. Descriptions of the field exploration program and the boring logs are presented in Appendix A, Field Explorations.

Laboratory Testing

Soil samples were obtained during drilling and taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for the determination of moisture content, fines content (material passing the U.S. No. 200 sieve), grain size distribution (sieve analyses), and Atterberg Limits. A description of the laboratory testing and the test results are presented in Appendix B, Laboratory Testing.

PREVIOUS SITE EVALUATIONS

In addition to the explorations completed as part of this evaluation, the logs of selected explorations from previous site evaluations in the project vicinity were reviewed. The logs of explorations from previous projects referenced for this study are presented in Appendix C, Exploration Logs from Previous Studies.

SITE CONDITIONS

Surface Conditions

The site is currently occupied by one office building surrounded by surface parking, access roads, and landscaped areas. The building consists of a two-story masonry office building that was constructed around the mid-1980s. We understand that landmark and significant trees are also present across the project site. The site slopes down from west to east, with a total change in elevation of up to approximately 88 feet.

Numerous buried utilities are located within and near the project site and within the public right-of-way (ROW) along the adjacent streets. These utilities include, but are not limited to, electrical, telecommunication, gas, overhead power, water, sanitary sewer, and storm drain.

Subsurface Conditions

GeoEngineers' understanding of subsurface conditions is based on the review of existing geotechnical information and the results of borings and monitoring wells drilled as part of this study. The approximate locations of the previous and recent explorations are presented in Figure 2.

Soils encountered at the site consist of relatively shallow fill overlying recent deposits and competent glacially consolidated soils. Interpreted subsurface conditions are presented in Cross Sections A-A′ through C-C′, Figures 3 through 5, respectively.

- The fill encountered at the site generally consists of loose to medium dense silty sand with variable gravel and cobble content. We understand that grading was completed on the site as part of construction of the existing building. The thickness of fill encountered at the site ranges up to approximately 19½ feet, with the deepest portion located towards the north end of the project site.
- The recent deposits encountered in the explorations typically consist of stiff to very stiff silt and clay with occasional sand interbeds and variable gravel content or medium dense to dense sand with variable silt and gravel. Recent deposits were observed below the fill or at the ground surface, where no fill was encountered. The recent deposits typically range in thickness up to 25 feet. However, in borings GEI-3 and GEI-6, the recent deposit layer is 56 and 61 feet thick, respectively. The thickness of recent deposits in the GEI-3 and GEI-6 locations is interpreted to reflect the lateral margins of alluvial deposits that infill the Sammamish River Valley east of the site and form part of the Redmond Alluvial Aquifer.
- Glacially consolidated soils were encountered below the fill and recent deposits, where present. Three glacially consolidated units were encountered in the explorations: cohesionless sand and gravel, till-like deposits, and cohesive silt and clay. The cohesionless sand and gravel soils consist of very dense sand and gravel with silt and occasional cobbles with isolated layers of silty sand with gravel. The till-like deposits consist of very dense silty sand with gravel and variable cobble and boulder content with isolated layers of hard clay. The cohesive silt and clay consists of very stiff to hard silt and clay layers. In general, the till-like deposits over the cohesive silt and clay soils and localized layers of cohesionless sand and gravel soils are present within the till-like soils or between the till-like soils and cohesive silt and clay soils. Where encountered in the borings, the till-like and cohesive silt and clay deposits extended to the depths explored.

While not encountered during drilling, occasional boulders have been observed in glacial soils on project sites with similar geology and may be present at this site.

Critical and Sensitive Area Designations

We reviewed the City of Redmond's online maps for critical and sensitive areas within the project site and vicinity, including maps of erosion, landslide and seismic hazard areas.

Based on our review, portions of the site are located within a mapped seismic hazard area. Mapped erosion and landslide hazard areas are located to the west of the project site.

Groundwater Conditions

The depth to groundwater was measured in the monitoring wells installed in selected borings, as well as monitoring wells installed on the site for a previous study. Automatic pressure transducers and data-loggers have been installed in six of the monitoring wells (not GEI-19) to observe the variability in groundwater levels seasonally and after significant rainfall events. The data-loggers will be left in the wells during the design phase of the project to provide additional data regarding groundwater level fluctuations.

The following table provides a summary of the monitoring wells and recent groundwater measurements at the site.

Well ID	Approximate Ground Surface Elevation (feet)	Approximate Top of Casing Elevation (feet)	Approximate Well Screen Elevation (feet)	Measured Groundwater Elevation (feet)	Range in Groundwater Elevations (feet)
GEI-1	86	85.4	14 to 24	76.7 (2/23/18) 77.6 (2/28/18) 77.8 (3/6/18) 77.7 (3/12/18) 78.3 (3/28/18) 77.7 (7/13/18) 78.0 (1/22/19) 77.2 (5/30/19	75.1 to 80.1
GEI-3	51	50.2	1 to 13	28.1 (3/2/18) 28.0 (3/6/18) 27.9 (3/12/18) 27.2 (3/28/18) 25.2 (7/13/18) 27.5 (1/22/19)	24.4 to 29.7
GEI-5	72	71.7	51 to 61	57.1 (3/12/18) 57.3 (3/28/18) 56.1 (7/13/18) 58.0 (1/22/19) 57.3 (5/30/19)	56.3 to 59.7
GEI-9	61	60.3	21 to 31	34.6 (3/12/18) 33.4 (3/28/18) 35.0 (7/13/18) 31.9 (1/22/19) 30.7 (5/30/19)	29.2 to 35.6
GEI-10	97	96.6	22 to 32	>96.6 (3/12/18)[1] >96.6 (3/28/18)[1] >96.6 (7/13/18)[1] >96.6 (1/22/19)[1] >96.6 (5/30/19)[1]	>96.6
GEI-11	86	85.1	36 to 46	>85.1 (3/12/18)[1] >85.1 (3/28/18)[1] >85.1 (7/13/18)[1] >85.1 (1/22/19)[1] >85.1 (5/30/19)[1]	>85.1
GEI-19	92	91.2	8 to 23	84.3 (1/22/19) 80.7 (5/30/19)	80.5 to 85.5
AMW-01	60	59.6	25 to 40	30.7 (12/15/17)[2] 32.7 (2/23/18) 31.4 (3/6/18) 31.2 (3/12/18) 29.6 (1/22/19) 27.1 (5/30/19)	-[3]

Well ID	Approximate Ground Surface Elevation (feet)	Approximate Top of Casing Elevation (feet)	Approximate Well Screen Elevation (feet)	Measured Groundwater Elevation (feet)	Range in Groundwater Elevations (feet)
AMW-04	96	95.2	69 to 84	90.5 (12/15/17)[2] 90.3 (2/23/18) 91.2 (3/6/18) 91.1 (3/12/18) 90.2 (1/22/19) 90.0 (5/22/19	-[3]

Notes:

[1] Artesian conditions present. Reading not representative because water flowing above casing.

[2] Groundwater reading reported in Aspect Consulting's draft report dated December 19, 2017.

[3] No range reported because monitoring wells were not equipped with automated pressure transducers.

Based on monitoring well data, conditions observed during drilling and data from monitoring wells in the project vicinity, we interpret three groundwater conditions at the site: (1) groundwater present in a shallow unconfined aquifer, (2) groundwater present in a deep confined aquifer, and (3) isolated perched groundwater present within relatively low permeability soils.

Groundwater present within the shallow unconfined aquifer was encountered in the granular fill and recent deposits. The monitoring wells in GEI-3, GEI-5, and GEI-9 are screened within the shallow aquifer.

Groundwater present with the deep confined aquifer is present within the glacially consolidated cohesionless sand and gravel deposits and thin sand layers within the till-like soils. The monitoring wells in GEI-1, GEI-10, GEI-11, and AMW-01 are screened within the deep confined aquifer. It should be noted that there is a significant confined groundwater head observed in these monitoring wells. The monitoring wells in GEI-10 and GEI-11 showed artesian conditions with the water level flowing above the monitoring well casing.

Perched groundwater is present locally in isolated layers within lower permeability glacially consolidated till-like deposits. The perched groundwater is not anticipated to represent a significant design or construction consideration. The monitoring well in AMW-04 is screened within the glacially consolidated till-like deposits and groundwater in this well is interpreted to be perched groundwater.

Groundwater levels are anticipated to vary as a function of location, precipitation, season and other factors. Additional groundwater measurements will be taken leading up to construction to assess seasonal variations in groundwater elevations.

CONCLUSIONS AND RECOMMENDATIONS

Summary

A summary of the primary geotechnical considerations is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- Portions of the site are located within a mapped seismic hazard area. The seismic hazard mapping is consistent with the explorations completed for this project and our liquefaction susceptibility analyses. Where present, potentially liquefiable soils are recommended to be mitigated using ground improvement or deep foundations.
- The site is designated as seismic Soil Profile Type F per the 2015 International Building Code (IBC) and American Society of Civil Engineers (ASCE) 7-16 due to the presence of potentially liquefiable soils below the building footprint. Given the combination of the presence of potentially liquefiable soils and the building period longer than 0.5 seconds, a site-specific seismic response analysis has been completed to determine the site-specific response spectrum. To mitigate potential seismic settlement, ground improvement or deep foundations are planned, where necessary, below the building footprint. Given the silty and variable nature of the fill and recent deposits, the liquefaction will be mitigated by transferring building loads to the bearing soil layer by means of removal and replacement with structural fill or through the use of ground improvement or deep foundations.
- The planned excavation will extend below the groundwater table in portions of the building, therefore temporary dewatering will be required during construction. Temporary dewatering can be completed using either vacuum wellpoints and/or sumps and pumps. The type of dewatering system and the system's configuration will depend on the type of shoring system implemented and on the contractor's preferences for completing excavation and construction of the below-grade portion of the building. The depth of the excavation, the type of shoring system, and the type of dewatering system design will influence the dewatering flow rates. Based on our review of conceptual building designs, for an excavation extending up to approximately 25 feet below site grades, we estimate flow rates on the order of 50 gallons per minute (gpm) or less.
- For design, temporary shoring for the site excavation is recommended to be completed using soldier pile and tieback shoring. Due to the shallow groundwater and variable soil layers at the site, soil nailing is not the preferred shoring system.
- Variable soils are present across the site and foundation support options vary depending on building locations and lowest finished floor elevation. In areas where the lowest finished floor elevation is located within the glacially consolidated soils, high capacity shallow and mat foundations can be used. In areas where the lowest finished floor elevation is located within fill and/or recent deposits, ground improvement or deep foundations will be required. Figure 6 presents the estimated elevation of the glacially consolidated soils across the building footprint (estimated top of bearing soils map).
- The lowest finished floor elevation will be located below the groundwater table in portions of the site. Permanent dewatering completed through collection of water in a below slab drainage system is anticipated and will allow for conventional basement walls and slabs-on-grade.
- Conventional slabs-on-grade are considered appropriate and should be underlain by a 6-inch-thick layer of clean crushed rock.

Our specific preliminary geotechnical recommendations are presented in the following sections of this report.

Earthquake Engineering

The site is designated as seismic Soil Profile Type F per the 2015 IBC and ASCE 7-16 due to the presence of potentially liquefiable soils below the building footprint. Given the combination of the presence of potentially liquefiable soils and the building period longer than 0.5 seconds, a site-specific seismic response analysis has been completed to determine the site-specific response spectrum. To mitigate potential static and seismic settlement, ground improvement or deep foundations are planned, where necessary, below the building footprint. Given the silty and variable nature of the fill and recent deposits, the static and seismic settlement potential will be mitigated by transferring building loads to the bearing soil layer by means of removal and replacement with structural fill or through the use of ground improvement or deep foundations

The results of the site-specific response analysis are included in Appendix E, Site-specific Response Analysis.

Liquefaction

Liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and some silts that are below the water table.

The results of our analyses indicate that the very loose to medium dense fill and recent deposit below the groundwater table have a low to moderate potential for liquefaction during a design earthquake event.

The evaluation of liquefaction potential is a complex procedure and is dependent on numerous site parameters, including soil grain size, soil density, site geometry, static stress, and the design ground acceleration. Typically, the liquefaction potential of a site is evaluated by comparing the cyclic stress ratio (CSR), which is the ratio of the cyclic shear stress induced by an earthquake to the initial effective overburden stress, to the cyclic resistance ratio (CRR), which is the soils resistance to liquefaction. Estimation of the CSR and the CRR were completed using empirical methods (Youd, et al. 2001).

Estimated ground settlement resulting from earthquake-induced liquefaction was analyzed using empirical procedures based on correlations from the standard penetration test (SPT) results (Tokimatsu and Seed 1987; Ishihara and Yoshimine 1992).

Our analyses indicate that during a design earthquake, settlement caused by liquefaction of the saturated fill and recent deposits beneath the foundation elevation could range up to more than 12 inches. Due to the magnitude of liquefaction-induced settlement, and the low risk of liquefaction in other portions of the site, the potential for differential settlement exists. Further, the potentially liquefiable fill soils represent unsuitable bearing for static conditions. Ground improvement is planned, where necessary, to mitigate the potential for differential settlement and to transfer the building loads through the potentially liquefiable soils to the underlying bearing soils.

Lateral Spreading

Lateral spreading involves lateral displacement of large, surficial blocks of soil as the underlying soil layer liquefies. Due to the distance to the Sammamish River, and that the liquefiable soils (where present below the building) will be mitigated through the use of ground improvement or deep foundations, the potential for lateral spreading is considered low for the project site.

Other Seismic Hazards

Due to the location of the site and the site's topography, the risk of adverse impacts resulting from seismically induced slope instability, differential settlement, or surface displacement due to faulting is considered low.

Seismic Design Information

A site-specific response analysis was completed to develop the risk-targeted site-specific maximum-considered earthquake (MCE_R) response spectrum (2 percent probability of exceedance in 50 years, 2,475-year return period) for use in the structural design of the Building X project per the ASCE 7-16 code and the 2015 IBC. This analysis was required because the site is classified as Site Class F because of liquefiable soils being present on site and the building having a fundamental period greater than 0.5 seconds.

Site specific geophysical testing consisting of shear wave velocity measurements and vibration monitoring was completed at the project site. The results of the geophysical testing is presented in Appendix D.

The recommended site-specific MCE_R response spectrum developed for the proposed Building X project is presented in Appendix E.

Aquifer Properties

A series of slug tests were conducted in three monitoring wells between December 12 and 13, 2018 as a basis for estimating hydraulic conductivity in the aquifer zones encountered at the site. Appendix F includes the slug test results and details of the slug testing method. Based on the current foundation design, the excavation and construction of Building X will likely only encounter groundwater within the shallow unconfined aquifer. The average value of hydraulic conductivity of the shallow unconfined aquifer estimated from the slug test results was 2.0x 100×10^{-5} centimeters per second, equivalent to 0.06 feet per day (ft/d).

The descriptions of the soils encountered during drilling and the measured water levels in the monitoring wells suggests that groundwater accumulates from seepage and infiltration of precipitation on the ground surface within the site and from areas higher in elevation to the west. Groundwater flow likely occurs above approximately 19 feet below the ground surface (bgs), in a shallow unconfined aquifer zone where the material is primarily sand and therefore likely more permeable than the silt encountered below. The aquifer zone thickness at monitoring well GEI-5, where the deepest portion of the Building X foundation is proposed, is assumed to be 10 feet based on the water table measured on January 22, 2019 and the depth of the underlying silt layer.

Aquifer thickness will change seasonally due to water table fluctuations and will likely increase in late winter through to late spring. Based on the average value of hydraulic conductivity estimated from the slug test results in GEI-5, the aquifer zone has a transmissivity (i.e., hydraulic conductivity multiplied by saturated thickness) of 0.6 ft^2/d.

Temporary Dewatering

The planned excavations are anticipated to extend below the groundwater table measured in monitoring wells across the site and temporary dewatering is anticipated to be required in order to complete the planned excavation. Temporary dewatering may be accomplished using a variety of means but the use of deep dewatering wells located around the perimeter of the excavation is not anticipated for this site because of the relatively low hydraulic conductivity values measured during slug testing. Vacuum wellpoints are likely a more suitable means of providing temporary dewatering and should be used in portions of the planned excavation where a lower drawdown (less than 20 feet) is required. Sumps and pumps can be used in areas where shallow drawdown, on the order of 5 feet or less, is required.

The temporary dewatering system should be designed to maintain the groundwater level at least 3 feet below the foundation subgrade elevation until the below slab drainage system is operational.

Most of the groundwater flow into the planned excavation is anticipated to be produced from the shallow unconfined aquifer. The temporary dewatering flow rates will depend on the location of the buildings, footprint area of the buildings, and lowest finished floor elevations. Based on our review of conceptual building designs, for an excavation extending up to approximately 25 feet below site grades, we estimate flow rates on the order of 50 gpm or less. GeoEngineers recommends that groundwater monitoring wells be installed near the excavation to monitor groundwater levels during construction. The purpose of these groundwater monitoring wells is to confirm that the dewatering system is performing as intended and to confirm that dewatering is functioning to reduce the potential for excessive buoyant pressures acting on the building until sufficient structural loads are present to resist buoyancy. The number and location of groundwater monitoring wells can be determined once the temporary dewatering plans are available.

GeoEngineers recommends that the dewatering system design be completed by the shoring contractor (design-build) and that GeoEngineers review the temporary dewatering plan once it is available.

Excavation Support

We understand that the planned building will have multiple below-grade levels that will follow the current ground surface contours and include modest excavations (less than 25 feet below current site grades). The subsurface conditions support the use of cantilever soldier pile or soldier pile and tieback walls with temporary dewatering for temporary excavation support. If sufficient space is available, temporary cut slopes can be implemented. Due to the variable groundwater and soil layers at the site, soil nailing is not the preferred shoring system. The following sections provide geotechnical design and construction recommendations for soldier pile and tieback walls.

We provide preliminary geotechnical design and construction recommendations for soldier pile and tieback walls below.

Excavation Considerations

The site soils may be excavated with conventional excavation equipment, such as trackhoes or dozers. It may be necessary to rip the glacially consolidated soils locally to facilitate excavation. The contractor should be prepared for occasional cobbles and boulders in the site soils. Likewise, the surficial fill may contain foundation elements and/or utilities from previous site development, debris, rubble and/or cobbles and boulders. We recommend that procedures be identified in the project specifications for measurement and payment of work associated with obstructions.

Soldier Pile and Tieback Walls

Soldier pile walls consist of steel beams that are concreted into drilled vertical holes located along the wall alignment, typically about 8 feet on center. After excavation to specified elevations, tiebacks are installed, if necessary. Once the tiebacks are installed, the pullout capacity of each tieback is tested, and the tieback is locked off to the soldier pile at or near the design tieback load. Tiebacks typically consist of steel strands that are installed into pre-drilled holes and then either tremie or pressure grouted. Timber lagging is typically installed behind the flanges of the steel beams to retain the soil located between the soldier piles. Geotechnical design recommendations for each of these components of the soldier pile and tieback wall system are presented in the following sections.

Soldier Piles

We recommend that soldier pile walls be designed using the earth pressure diagram presented in Figure 7 are for full-height cantilever soldier pile walls and soldier pile walls with single or multiple levels of tiebacks, and the pressures represent the estimated loads that will be applied to the wall system for various wall heights.

The earth pressures presented in Figure 7 include the loading from traffic or construction surcharge. Other surcharge loads, such as cranes, construction equipment or construction staging areas, should be considered by GeoEngineers on a case-by-case basis. No seismic pressures have been included in Figure 7 because the earth pressures shown in Figure 7 are for temporary shoring.

We recommend that the embedded portion of the soldier piles be at least 2 feet in diameter and extend a minimum distance of 10 feet below the base of the excavation to resist “kick-out.” The axial capacity of the soldier piles must resist the downward component of the anchor loads and other vertical loads, as appropriate. We recommend using an allowable end bearing value of 20 kips per square foot (ksf) for piles supported on the glacially consolidated soils. The allowable end bearing value should be applied to the base area of the drilled hole into which the soldier pile is concreted. This value includes a factor of safety of about 2.5. The allowable end bearing value assumes that the shaft bottom is cleaned out immediately prior to concrete placement. If necessary, an allowable pile skin friction of 0.75 ksf may be used on the embedded portion of the soldier piles, if terminated into glacially consolidated soils, to resist the vertical loads.

Lagging

The following table presents recommend lagging thicknesses (roughcut) as a function of soldier pile clear span and depth.

	Recommended Lagging Thickness (roughcut) for clear spans of:					
Depth (feet)	**5 feet**	**6 feet**	**7 feet**	**8 feet**	**9 feet**	**10 feet**
0 to 25	2 inches	3 inches	3 inches	3 inches	4 inches	4 inches
25 to 100	3 inches	3 inches	3 inches	4 inches	4 inches	5 inches

Lagging should be installed promptly after excavation, especially in areas where perched groundwater is present or where clean sand and gravel soils are present and caving soils conditions are likely. The workmanship associated with lagging installation is important for maintaining the integrity of the excavation.

The space behind the lagging should be filled with soil as soon as practicable. Placement of this material will help reduce the risk of voids developing behind the wall and damage to existing improvements located behind the wall.

Lean concrete is a suitable option for the use of backfill behind the walls. Lean concrete will reduce the volume of voids present behind the wall. Alternatively, lean concrete may be used for backfill behind the upper 15 to 20 feet of the excavation to limit caving and sloughing of the upper soils, with on-site soils used to backfill the voids for the remainder of the excavation. Based on our experience, the voids between each lean concrete lift are sufficient for preventing the buildup of hydrostatic pressure behind the wall.

Tiebacks

Tieback anchors can be used for wall heights where cantilever soldier pile walls are not cost-effective. Tieback anchors should extend far enough behind the wall to develop anchorage beyond the "no-load" zone and within a stable soil mass. The anchors should be inclined downward at 15 to 25 degrees below the horizontal. Corrosion protection will not be required for the temporary tiebacks.

Centralizers should be used to keep the tieback in the center of the hole during grouting. Structural grout or concrete should be used to fill the bond zone of the tiebacks. A bond breaker, such as plastic sheathing, should be placed around the portion of the tieback located within the no-load zone if the shoring contractor plans to grout both the bond zone and unbonded zone of the tiebacks in a single stage. If the shoring contractor does not plan to use a bond breaker to isolate the no-load zone, GeoEngineers should be contacted to provide recommendations.

Loose soil and slough should be removed from the holes drilled for tieback anchors prior to installing the tieback. The contractor should take necessary precautions to minimize loss of ground and prevent disturbance to previously installed anchors and existing improvements in the site vicinity. Holes drilled for tiebacks should be grouted/filled promptly to reduce the potential for loss of ground.

Tieback anchors should develop anchorage in the glacially consolidated soils. We recommend that spacing between tiebacks be at least three times the diameter of the anchor hole to minimize group interaction. We recommend a design load transfer value between the anchor and soil of 4 kips per foot for glacially consolidated soils and 1.5 kips per foot for fill and recent deposits.

The tieback anchors should be verification- and proof-tested to confirm that the tiebacks have adequate pullout capacity. The pullout resistance of tiebacks should be designed using a factor of safety of 2. The pullout resistance should be verified by completing at least two successful verification tests in each soil type and a minimum of four total tests for the project. Each tieback should be proof-tested to 133 percent of the design load. Verification and proof tests should be completed as described in Appendix G, Ground Anchor Load Tests and Shoring Monitoring Program.

The installation of tiebacks located within fine-grained soils (silts and clays) should be drilled with care. The air used during drilling to flush the cuttings should be carefully controlled so that no choke points occur, and the air is not forced into the soil formation. The post-grouting of tiebacks, if implemented, should also be completed with care to avoid displacements within the soil formation.

The tieback layout and inclination should be checked to confirm that the tiebacks do not interfere with adjacent buried utilities. The City of Redmond minimum clearances between ground anchors and existing utilities should be maintained.

Drainage

Drainage for soldier pile and lagging walls is achieved through seepage through the timber lagging. Seepage flows at the bottom of the excavation should be contained and controlled to prevent loss of soil from behind the lagging. Drainage should be provided for permanent below-grade walls as described below in the "Below-Grade Walls" section of this report.

Construction Considerations

Temporary casing or drilling fluid may be required to install the soldier piles and tiebacks where:

- Loose fill is present;
- The native soils do not have adequate cementation or cohesion to prevent caving or raveling; and/or
- Groundwater is present.

GeoEngineers should be allowed to observe and document the installation and testing of the shoring to verify conformance with the design assumptions and recommendations.

Shoring Wall Performance

Shoring walls typically move on the order of 0.1 to 0.2 percent of H, where H is the vertical distance between the existing ground surface and the base of excavation.

The deflections and settlements are usually highest at the excavation face and decrease to negligible amounts beyond a distance behind the wall equal to the height of the excavation. Localized deflections may exceed the above estimates and may reflect local variations in soil conditions (such as around utilities or areas of thick fill) or may be the result of the workmanship used to construct the shoring wall. Given that some movement is expected, existing improvements located adjacent to the shoring system will also experience movement. The deformations discussed above are not likely to cause structural damage to structurally sound existing improvements; however, some cosmetic damage should be expected (for instance, cracks in drywall finishes; widening of existing cracks; minor cracking of slabs-on-grade/hardscapes; cracking of sidewalks, curbs/gutter, and pavements/pavement panels; etc.). For this reason, it is important to complete a pre-construction survey and photo documentation of existing buildings and improvements prior to shoring construction. Refer to Appendix G for more detailed recommendations for shoring monitoring and preconstruction survey.

Gravity Walls

We understand that gravity walls will be used in areas of the project site where limited cuts (less than 5 feet in height) will be completed. The gravity walls are anticipated to consist or gabions or rockeries.

Gabions

Gabion cribbing consists of wire mesh baskets that are fastened together with spiral binders. Gabion baskets are typically filled with stone, and can be rounded cobble, or more commonly, angular stone ranging from 3 to 8 inches.

We understand that due to the proximity of the gabion walls to existing trees that are to be protected, limited excavation behind the wall is anticipated. Because of this, we do not anticipate a zone of excavation for reinforcement. The walls are anticipated to have a backslope of up to 3H:1V (horizontal to vertical) and a foreslope of up to 4H:1V.

We anticipate the walls will need to support a live load of up to 50 psf behind the backslope.

An overview of recommendations for the gabion gravity walls is provided in the following sections.

Subgrade Preparation

Before placing wall elements, the wall subgrade should be prepared as described in the "Earthwork" section of this report. We recommend that the gabions be founded on a minimum of 6 inches of Mineral Aggregate Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14.

GeoEngineers should evaluate wall subgrade prior to placement of the leveling pad. If excessively soft soils are encountered below the subgrade for gabion walls, the soft soils should be removed and replaced with properly compacted structural fill, prepared as described in the "Structural Fill" section of the report. Organic soils, if encountered, should be removed from below both the gabion subgrade zone.

Gabion Wall Embedment

The base of the walls and finished grade in front of the walls should be in accordance with the elevations in the project plans. A minimum embedment of 1 foot is required for the gabion gravity walls.

Gabion Wall Drainage

Gabion gravity walls should be designed with and 6- to 12-inch wide zone of drain rock between the gabion cribbing and retained soil. The drain rock should consist of Mineral Aggregate Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14, and a perforated pipe with a diameter of at least 4 inches should be installed at the base of the drain rock. A non-woven geotextile separator fabric, such as Mirafi 140N or similar should be placed between the drain rock zone and the adjacent and underlying retained and foundation soils.

Rockeries

We understand that rockeries may be used for grade transitions at the site. If rockeries are used at the site, it should be understood that the primary purpose of a rockery is to protect the soil from erosion and raveling, while providing limited soil retention. Rockeries may be used in both cut and fill areas. Rockeries with a 1H:5V to 1H:6V batter and maximum 3H:1V backslope should be limited to 4 feet of exposed height in cuts made in fill deposits. The height is measured as the vertical distance from the ground surface in front of the toe of the rockery to the ground surface behind the top of the rockery.

The base of rockeries should be embedded at least 1 foot below the adjacent ground surface. The rockery should be supported on firm, undisturbed glacially consolidated soils or compacted structural fill. The rockery should be constructed using rock sizes and procedures specified in City of Redmond Standard Detail 909.

A minimum 6-inch diameter perforated drain should be embedded in the backfill at the base of the rockery. The drain should be tightlined to the storm drainage system.

Foundation Support

The soils at the foundation elevation for the planned building vary across the site. For foundations that bear on competent glacially consolidated soils, shallow spread or mat foundations are may be used. For foundations that will bear on fill or recent deposits, deep foundations or shallow foundations bearing on improved ground are recommended.

Shallow Foundations

Based on the data obtained from the borings completed at the site and the anticipated depth of excavation, portions of the foundation levels are anticipated to extend into the glacially consolidated soils. We recommend that the buildings be supported on shallow spread or mat foundations bearing on the very dense or hard glacially consolidated soils. Where recent deposits or fill soils are present at foundation subgrade elevation, shallow spread or mat foundations should bear on improved ground extending down to very dense or hard glacially consolidated soils.

Allowable Bearing Pressure

For foundations constructed as recommended in this report, we recommend using an allowable bearing pressure of 12 ksf for shallow spread or structural mat foundations bearing on the dense to very dense/hard glacially consolidated soils located below existing site grades. We recommend using an allowable bearing pressure of 6 ksf for shallow spread or structural mat foundations bearing on improved ground extending down to very dense or hard glacially consolidated soils. The allowable soil bearing pressure applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads. The allowable soil bearing pressures are net values.

Modulus of Subgrade Reaction

For mat foundations designed as a beam on an elastic foundation, a modulus of subgrade reaction of 83 pounds per cubic inch (pci) may be used for the portion of the structural mat foundation bearing on very dense or hard glacially consolidated soils. A modulus of subgrade reaction of 42 pci may be used for the portion of the structural mat foundation bearing on improved ground.

Settlement

Provided that all loose soil is removed, and that the subgrade is prepared as recommended under "Construction Considerations" below, we estimate that the total settlement of structural mat foundations will be about 1 inch or less. The settlements will occur rapidly, essentially as loads are applied. Differential settlements between footings could be half of the total settlement. Note that smaller settlements will result from lower applied loads.

Lateral Resistance

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the shallow foundations. For shallow foundations supported on native soils, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 400 pounds per cubic foot (pcf) (triangular distribution). These values are appropriate for foundation elements that are poured directly against undisturbed glacially consolidated soils or surrounded by structural fill.

The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

Construction Considerations

The site soils are susceptible to softening from water or construction traffic. We recommend that the contractor be prepared to pour a mud mat consisting of lean concrete across the exposed foundation subgrade to protect it from softening during wet weather conditions or where groundwater seepage is present.

If soft areas are present at the footing subgrade elevation, the soft areas should be removed and replaced with lean concrete or structural concrete at the direction of GeoEngineers.

We recommend that GeoEngineers observe the condition of all subgrade areas to evaluate whether the work is completed in accordance with our recommendations and whether the subsurface conditions are as anticipated.

Deep Foundations

Deep foundations can be used in the eastern portion of the site because potentially liquefiable soils will be located below the lowest finished floor elevation of the planned building. Deep foundation options consist of either augercast piles or drilled shafts.

Drilled Shafts

Drilled shafts consist of steel reinforcement cages that are concreted into drilled vertical holes located below foundations or building walls. Drilled shafts are typically drilled with an open flight auger and backfilled with structural concrete via tremie pipe when below the groundwater table after the reinforcement is set in the hole. Core barrels are typically used to drill through large obstructions and bucket augers are typically used when caving conditions are encountered below the groundwater table. The driller may use a steel casing if caving conditions are encountered.

Axial Capacity

We evaluated axial capacities for 4-foot and 5-foot diameter drilled shafts per the 2015 American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications manual. We recommend that drilled shafts be embedded at least 15 to 20 feet into the glacially consolidated soils. For static conditions, we recommend that axial capacities for drilled shafts consist of factored side resistance along the total length of the shaft and factored end bearing at the base of shaft in the glacially consolidated soils. For seismic liquefied soil conditions, we recommend that axial capacities for drilled shafts consist of factored side resistance and end bearing in the non-liquefied glacially consolidated soils and should also consider downdrag from liquefaction-induced settlement from the design earthquake. We recommend using strength parameters for static conditions and extreme parameters for seismic conditions. Preliminary recommendations for factored side resistance and end bearing are presented in the following table. Axial capacities assume 1 inch of shaft settlement. The capacities in the table below can be used for various sized drilled shafts.

Soil Type	Static Conditions		Seismic Conditions		
	Side Resistance (ksf)	**End Bearing (ksf)**	**Side Resistance (ksf)**	**End Bearing (ksf)**	**Downdrag (ksf)**
Liquefiable Fill/Recent Deposits	0	n/a	0	n/a	-0.45
Glacially Consolidated Soils	1.5[1]	30[1]	2.5[2]	60[2]	n/a

Notes:

[1] For static conditions (Strength Limit State), a resistance factor of 0.55 was used for side resistance and 0.5 for end bearing

[2] For seismic conditions (Extreme Limit State), a resistance factor of 1.0 was used for side resistance and end bearing

The structural characteristics of shaft materials and structural connections may impose limitations on shaft capacity and should be evaluated by the structural engineer. Full length steel reinforcing will be needed for shafts subjected to uplift loads.

Full depth temporary casing or drilling slurries may be required to maintain shaft sidewall stability when drilling through loose/granular soil materials and below the groundwater table as they can be prone to caving if left unsupported.

Drilled shafts should be excavated with equipment that reduces the loose cuttings or slough at the bottom of the drilled hole. Slough and loose cuttings should be removed from the hole prior to placing concrete. Where “wet” construction methods are used, it will be necessary to use tremie methods for placement of concrete. Nondestructive testing of shafts using Cross-Hole Sonic Logging (CSL) is recommended for all drilled shafts.

We recommend that the shaft installation adhere to shaft installation guidelines, construction requirements, integrity testing, and acceptance criteria are provided in the Washington State Department of Transportation (WSDOT) Standard Specification 6-19 Shafts and FHWA-NHI-10-016 (Brown et al. 2010).

Augercast Piles

Augercast piles are constructed using a continuous-flight, hollow-stem auger attached to a set of leads supported by a crane or installed with a fixed-mast drill rig. The first step in the pile casting process consists of drilling the auger into the ground to the specified tip elevation of the pile. Grout is then pumped through the hollow stem during steady withdrawal of the auger, replacing the soils on the flights of the auger. The final step is to install a steel reinforcing cage and typically a center bar into the column of fresh grout. One benefit of using augercast piles is that the auger provides support for the soils during the pile installation process, thus eliminating the need for temporary casing or drilling fluid.

Preliminary recommendations for augercast piles are provided in the following sections.

Construction Considerations

The augercast piles should be installed using a continuous-flight, hollow-stem auger. Given the distinct contrast in stiffness between the fill and recent deposits and the underlying glacially consolidated soils and the need to develop pile capacity from the glacially consolidated soils, it is important that the piles achieve a consistent embedment into the glacially consolidated soils. To confirm that the piles are consistently embedded into the glacially consolidated soils, we recommend that the contractor use drilling equipment capable of measuring and displaying drill pressure and crowd speed during augercast pile installation. These measurements can be used as an indication of the transition from loose fill soils or recent deposits to denser glacially consolidated soils, which can be used to estimate pile embedment in glacially consolidated soils. Production piles located near one of the geotechnical borings completed for this project should be installed at the beginning of pile construction to calibrate the drill pressure and crowd speed output for the fill, recent deposits and the glacially consolidated soils. This process will provide the required information to determine whether the piles have been installed to an appropriate length and will eliminate the need for static pile load testing.

As is standard practice, the pile grout must be pumped under pressure through the hollow stem as the auger is withdrawn. Maintenance of adequate grout pressure at the auger tip is critical to reduce the potential for encroachment of adjacent native soils into the grout column. The rate of withdrawal of the auger must remain constant throughout the installation of the piles to reduce the potential for necking of the piles. Failure to maintain a constant rate of withdrawal of the auger should result in immediate rejection of that pile. Reinforcing steel for bending and uplift should be placed in the fresh grout column as soon as possible after withdrawal of the auger. Centering devices should be used to provide concrete cover around the reinforcing steel.

The contractor should adhere to a waiting period of at least 12 hours between the installation of piles spaced closer than 8 feet, center-to-center. This waiting period is necessary to avoid disturbing the curing concrete in previously cast piles.

Grout pumps must be fitted with a volume-measuring device and pressure gauge so that the volume of grout placed in each pile and the pressure head maintained during pumping can be observed. A minimum grout line pressure of 100 pounds per square inch (psi) should be maintained. The rate of auger withdrawal should be controlled during grouting such that the volume of grout pumped is equal to at least 115 percent of the theoretical pile volume. A minimum head of 10 feet of grout should be maintained above the auger tip during withdrawal of the auger to maintain a full column of grout and to prevent hole collapse.

A qualified geotechnical engineer should observe the drilling operations, monitor grout injection procedures, record the volume of grout placed in each pile relative to the calculated volume of the hole, and evaluate the adequacy of individual pile installations.

Axial Capacity

Axial pile load capacity at this site is primarily developed from end bearing in the very dense glacially consolidated soils, with some additional capacity attributed to side frictional resistance in the very dense glacially consolidated soils. Uplift pile capacity will also be developed from side frictional resistance in these soils.

For planning purposes, we recommend assuming 3-foot-diameter augercast piles that are embedded 15 to 20 feet into glacially consolidated soils. We recommend that the assumed embedment depth be confirmed by the general contractor to ensure that the selected specialty contractor's equipment can achieve the design embedment depth. Allowable augercast pile capacities were evaluated based on Allowable Stress Design (ASD) and are for combined dead plus long-term live loads and may be increased by one-third when considering design loads of short duration such as seismic forces. The static allowable capacities are based on the strength of the supporting soils and include a factor of safety of 3 for end bearing and 2 for shaft friction. The seismic allowable capacities assume no side resistance in the liquefiable soils and include a factor of safety of 2 for ending bearing and 1.5 for side resistance in the glacially consolidated soils. Preliminary recommendations for allowable side resistance and end bearing are presented in the following table. Axial capacities assume 1 inch of pile settlement. The capacities in the table below can be used for various sized augercast piles.

Soil Type	Static Conditions		Seismic Conditions		
	Side Resistance (ksf)	**End Bearing (ksf)**	**Side Resistance (ksf)**	**End Bearing (ksf)**	**Downdrag (ksf)**
Liquefiable Fill/Recent Deposits	0	n/a	0	n/a	-0.45
Glacially Consolidated Soils	1.5[1]	50[1]	3.0[2]	100[2]	n/a

Notes:

[1] For static conditions, a factor of safety of 2 was used for side resistance and 3 for end bearing

[2] For seismic conditions, a factor of safety of 1 was used for side resistance and 1.5 for end bearing

The capacities apply to single piles. If piles are spaced at least three pile diameters on center, as recommended, no reduction of axial capacity for group action is needed. The structural characteristics of pile materials and structural connections may impose limitations on pile capacities and should be evaluated by the structural engineer.

The structural characteristics of shaft materials and structural connections may impose limitations on shaft capacity and should be evaluated by the structural engineer. Full length steel reinforcing will be needed for shafts subjected to uplift loads.

Lateral Shaft Capacity

The shafts should be designed to resist lateral dead and live loads, as necessary. GeoEngineers can provide lateral resistance values or lateral design during the design phase once the preferred foundation type is selected.

Ground Improvement

Rigid inclusions are the preferred ground improvement system for support of foundation where the foundation subgrade elevation is located in the fill/recent deposits and the thickness of fill/recent deposits is more than is economical to use removal and replacement with structural fill.

The purpose of ground improvement is to mitigate potential static and/or seismic induced settlement/deformation from the fill and recent deposits. The benefits of ground improvement for this site include:

- Ground improvement will allow for conventional shallow foundations which are anticipated to result in more efficient and more cost-effective construction;
- Ground improvement will mitigate the potential static settlement resulting from consolidation of the near surface cohesive soils under the building loads; and
- Ground improvement will mitigate potential seismic induced settlements/deformation resulting from liquefaction of the fill and recent deposits by transferring the building loads to the non-liquefiable glacially consolidated soils.

Where ground improvement is used, an allowable bearing pressure of 6 ksf is recommended. The following section provides a general description of rigid inclusions.

Rigid Inclusions

Rigid inclusions consist of lean concrete columns that extend down to the dense to very dense/hard glacially consolidated soils below the building foundation elements on a variable grid pattern. The design concept for rigid inclusions consists of reinforcing compressible or liquefiable fill and recent deposits to transfer building loads to the more competent glacially consolidated soils. The purpose of the rigid inclusions placed in a grid pattern is to provide a significantly higher strength material capable of dissipating building loads in a less concentrated manner and to provide a 'block' of a composite soil and lean concrete material that will reduce the potential for differential settlement. Reinforcement consisting of a center bar may be added depending on site specific soil conditions and performance requirements.

Advantages with using rigid inclusions include: (1) lean concrete columns are more economical than augercast piles (shorter length, limited reinforcement, and allows for the use of conventional spread footings/slabs-on-grade), (2) minimal disturbance of adjacent structures and utilities during installation, and (3) lower level of construction noise (i.e. no pile driving) and subsequent lower impact to nearby businesses and residences during construction.

Rigid inclusions are constructed using similar techniques for installing drilled shafts or augercast piles. Where augercast methods are used, the first step in the rigid inclusion casting process consists of drilling the auger into the ground to the specified tip elevation of the column. Grout is then pumped into the hole using a tremie pipe. For drilled shaft installation methodology, the shaft is drilled to the specified tip elevation, the bottom of the shaft is then cleaned out, and lean concrete is placed into the drilled shaft. Measures to maintain shaft stability, such as the use of a water head, polymer slurry, or temporary casing may be required depending on the ground conditions. Where water is present in the drilled shaft, the concrete should be placed using a tremie pipe.

The layout and design of the rigid inclusions will be completed once the building design has been finalized. For preliminary design and pricing purposes, we provide the following preliminary design information for ground improvement:

- Eighteen- to 24-inch-diameter rigid inclusions;
- Use a 5- to 7-foot triangular rigid inclusion spacing; and

- Rigid inclusions will extend from the bottom of foundation elevation to through the fill and recent deposits into competent glacially consolidated soils.

GeoEngineers can assist the project team with preparation of the ground improvement plan and specifications once the foundation layout and building loads have been finalized.

Working Pad

Due to the nature of the existing soils, a working pad may be necessary to support equipment that will be used to install either deep foundations or ground improvement. We recommend that the working pad consist of a woven geotextile (Mirafi 600X or equivalent) with 12 to 18 inches of quarry spalls or permeable ballast material placed over the geotextile. Along the perimeter of the site, the working pad thickness can be reduced to 6 inches provided these areas are not required to support the ground improvement equipment. The quarry spalls shall meet the requirements of Quarry Spalls, WSDOT Standard Specification 9-13.6. Permeable ballast shall meet the requirements of WSDOT Standard Specification 9-03.9(2). The quarry spalls/permeable ballast shall be placed in loose lifts with a maximum thickness of 1 foot and compacted by heavy track-mounted equipment or a vibratory roller. The working pad thickness and material should be reviewed with the ground improvement/foundation specialty contractor to confirm or modify the design.

Slab-on-Grade Floors

Slabs-on-grade floors with below-slab drainage are anticipated for Building X. The following sections provide design recommendations for subgrade preparation, slab-on-grade design parameters, and below-slab drainage.

Subgrade Preparation

Exposed subgrade should be evaluated after site grading is complete. Proof-rolling with heavy, rubber-tired construction equipment should be used for this purpose during dry weather and if access is practical. Probing should be used to evaluate the subgrade during periods of wet weather or if access is not feasible for construction equipment. Exposed soil should be firm and unyielding, and without significant groundwater. Disturbed areas should be recompacted if possible or removed and replaced with compacted structural fill.

The site should be rough graded to approximately 1 foot above slab subgrade elevation prior to foundation construction in order to protect the slab subgrade soils from deterioration from wet weather or construction traffic. After the foundations and below-slab drainage system/below-slab utilities have been constructed, the remaining soils can be removed to final subgrade elevation followed by immediate placement of the capillary break material.

Design Parameters

Conventional slabs may be supported on-grade, provided the subgrade soils are prepared as recommended in the, “Subgrade Preparation” section above. We recommend that the slab be founded on either undisturbed glacially consolidated soils or on structural fill placed over the undisturbed glacially consolidated soils. For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 250 pci may be used for subgrade soils prepared as recommended.

In areas where the finished floor elevation is above the groundwater table, we recommend that the slab-on-grade floors be underlain by a 6-inch-thick capillary break consisting of material meeting the requirements of Mineral Aggregate Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.16.

In areas where the finished floor elevation is below the groundwater table, we recommend that the slab-on-grade floors be underlain by a 12-inch-thick capillary break consisting of material meeting the requirements of Mineral Aggregate Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.16. The capillary break should be underlain by nonwoven geotextile meeting the requirements of construction geotextile for underground drainage, WSDOT Standard Specification 9-33, such as Mirafi 140N (or approved equivalent). The purpose of the nonwoven geotextile is to provide separation between the native soils and the open graded capillary break material and to prevent the migration of fines and sediment within the capillary break material over time. GeoEngineers can assist the project team in assessing where the slab-on-grade is located below the groundwater table once the lowest finished floor elevation has been finalized.

Provided that loose soil is removed, and the subgrade is prepared as recommended, we estimate that slabs-on-grade will not settle appreciably.

Below-Slab Drainage

We recommend installing a below-slab drainage system to remove water from below the slab-on-grade. We recommend that the below-slab drainage system include an interior perimeter drain and three to four longitudinal drains. GeoEngineers should review the conceptual layout of the foundation drainage system to confirm that the intent of the below-slab drainage system is met prior to final design of the system.

The drainage pipe should be perforated with two rows of ½-inch holes spaced 120 degrees apart and at 4 inches on center. The below-slab drainage system trenches should be backfilled with Mineral Aggregate Type 22 (¾-inch crushed gravel) or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.16, or an alternative approved by GeoEngineers. The Type 22 or Type 5 material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, WSDOT Standard Specification 9-33. The below-slab drainage system pipes should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger-diameter pipe will allow for easier maintenance of drainage systems.

If no special waterproofing measures are taken, leaks or seepage may occur in localized areas of the below-grade portion of the building, even if the recommended wall drainage and below-slab drainage provisions are constructed. If leaks or seepage is undesirable, below-grade waterproofing should be specified.

Below-Grade Walls

Permanent Below-Grade Walls

Permanent below-grade walls should be designed for the same earth pressures (including surcharge pressures where applicable) as the adjacent temporary shoring walls and should also include a seismic load acting over the height of the wall equal to 8H psf, where H is the height of the wall in feet. Foundation surcharge loads, traffic surcharge loads, and the slab-on-grade load for the adjacent building, should be incorporated into the design of the below-grade walls using the surcharge pressures presented in Figure 8. Other surcharge loads, such as from construction equipment or construction staging areas, should be considered on a case-by-case basis. We can provide the lateral pressures from these surcharge loads as the design progresses.

Soil pressures recommended above assume that wall drains will be installed to prevent the buildup of hydrostatic pressure, as described below in the "Drainage" section of this report, and will be tied to permanent drains to remove the water in suitable discharge points.

Other Cast-in-Place Walls

Conventional cast-in-place walls will be necessary for basement walls constructed using open cuts and for small retaining structures located on-site. The lateral soil pressures acting on conventional cast-in-place subsurface walls will depend on the nature, density and configuration of the soil behind the wall and the amount of lateral wall movement that can occur as backfill is placed.

For walls that are free to yield at the top at least 0.1 percent of the height of the wall, soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing. Assuming that the walls are backfilled and drainage is provided as outlined in the following paragraphs, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (triangular distribution), while non-yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 55 pcf (triangular distribution). For seismic loading conditions, a rectangular earth pressure equal to 8H psf (where H is the height of the wall in feet) should be added to the active/at-rest pressures. Other surcharge loading should be applied as appropriate. Lateral resistance for conventional cast-in-place walls can be provided by frictional resistance along the base of the wall and passive resistance in front of the wall in accordance with the "Lateral Resistance" section earlier in this report.

The above soil pressures assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed in the paragraphs below.

Drainage

Drainage behind the permanent below-grade walls is typically provided using prefabricated drainage board attached to the shoring walls. Weep pipes that extend through the permanent below-grade wall should be installed around the perimeter of the building at the footing elevation. The weep pipes should have a minimum diameter of 4 inches. The weep pipes through the permanent below-grade wall should be spaced no more than 20 feet on center and should be hydraulically connected to the sump. These weep pipes may be designed for a hard connection to the perimeter drains discussed above in the "Below-Slab Drainage" section of this report.

The earth pressures for permanent below-grade walls assume that adequate drainage is provided behind the wall. Prefabricated geocomposite drainage material, such as Aquadrain 15X, should be installed vertically to the face of the lagging/shotcrete. The Aquadrain 15X drainage material should terminate at the base of the shoring wall into a base drain product such as Aquadrain 100DB. The weep pipes that penetrate the basement wall should be located in the base drain layer. For soldier pile or soil nail shoring walls, the drainage material should be installed on the excavation side of the lagging/shotcrete facing, with the fabric adjacent to the lagging/shotcrete facing.

Full wall face coverage is recommended to minimize seepage and/or wet areas at the face of the permanent wall. Full wall face coverage should extend from the weep pipe elevation up to about 3 to 5 feet below the top of the wall to reduce the potential for surface water to enter the wall drainage system. Although the use of full wall face coverage will reduce the likelihood of seepage and/or wet areas at the face of the permanent wall, the potential still exists for these conditions to occur. If this is a concern, waterproofing should be specified.

Positive drainage should be provided behind cast-in-place retaining walls by placing a minimum 2-foot-wide zone of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.16, with the exception that the percent passing the U.S. No. 200 sieve is to be less than 3 percent. A perforated drainpipe should be placed near the base of the retaining wall to provide drainage. The drainpipe should be surrounded by a minimum of 6 inches of Mineral Aggregate Type 22 (¾-inch crushed gravel) or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.16, or an alternative approved by GeoEngineers. The Type 22 or Type 5 material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, WSDOT Standard Specification 9-33. The wall drainpipe should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger-diameter pipe will allow for easier maintenance of drainage systems.

Earthwork

Clearing and Site Preparation

Construction of the building will require demolition of existing facilities, clearing and stripping. We expect that there will be demolition of existing structures, asphalt pavement, curbs, light poles, and utilities. Existing utilities should be removed from the building footprint and be rerouted if needed.

Areas to be developed or graded should be cleared of surface and subsurface deleterious matter including debris, shrubs, trees and associated stumps and roots. Graded areas should be stripped of organic materials and topsoil. Based on our explorations and site observations, we estimate that stripping depths will be on the order of 3 to 12 inches to remove vegetation or topsoil within existing landscaped or vegetated areas. Greater stripping depths might be needed in more densely vegetated areas and where large trees exist.

The stripped organic soils can be stockpiled and used later for landscaping purposes or may be spread over disturbed areas following completion of grading. If spread out, the organic strippings should be placed in a layer less than 1-foot thick, should not be placed on slopes greater than 3H:1V and should be track-rolled to a uniformly compacted condition. Materials that cannot be used for landscaping or protection of disturbed areas should be removed from the project site.

Subgrade Preparation

The exposed subgrade in structure and hardscape areas should be evaluated after site excavation is complete. Disturbed areas below slabs and foundations should be recompacted if the subgrade soil consists of granular material. If the subgrade soils consist of disturbed soils, it will likely be necessary to remove and replace the disturbed soil with structural fill unless the soil can be adequately moisture-conditioned and compacted.

Structural Fill

Fill placed to support structures, placed behind retaining structures, and placed below pavements and sidewalks will need to be specified as structural fill as described below:

- If structural fill is necessary beneath building foundations designed for an allowable bearing pressure of 12 ksf, the fill should consist of controlled density fill (CDF) or structural concrete. For foundations designed for an allowable bearing pressure of 6 ksf, structural fill placed below building foundations should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14 or Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.14.
- Structural fill placed behind cast-in-place retaining walls should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed within utility trenches should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed around below slab drains or cast-in-place wall drains should meet the requirements of Mineral Aggregate Type 5 (1-inch washed gravel) or Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14, with the exception that the percent fines be less than 3 percent.
- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.14.
- Structural fill used to raise site grades, to place below slabs-on-grade, and to place below foundations in areas where ground improvement is planned should consist of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14. However, if earthwork occurs during the normally dry months (June through September) on-site sandy soils that are properly moisture conditioned, that are free of debris, organics, and rubble, and that can be properly compacted may be used as structural fill in these areas. It may be possible to use on-site sandy soils during wet weather for areas requiring 90 percent compaction provided the earthwork contractor implements good wet weather techniques and drier soils are used; however, we recommend Type 17 be specified for planning/bidding purposes.

On-site Soils

The on-site soils are moisture-sensitive and generally have natural moisture contents higher than the anticipated optimum moisture content for compaction. As a result, the on-site soils will likely require moisture conditioning in order to meet the required compaction criteria during dry weather conditions and will not be suitable for reuse during wet weather. Furthermore, most of the fill soils required for the project have specific gradation requirements, and the on-site soils do not meet these gradation requirements. If the contractor wants to use on-site soils for structural fill, GeoEngineers can evaluate the on-site soils for suitability as structural fill, as required.

Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 1 foot in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to the following criteria:

- Structural fill placed in building areas (supporting foundations or slab-on-grade floors) should be compacted to at least 95 percent of the maximum dry density (MDD) estimated in general accordance with ASTM International (ASTM) Standard Practices Test Method D 1557.
- Structural fill in new pavement and hardscape areas should be compacted to at least 90 percent of the MDD, except that the upper 2 feet of fill below final subgrade level should be compacted to at least 95 percent of the MDD per ASTM D-1557.
- Structural fill placed in utility trenches should be compacted to at least 95 percent of the MDD per ASTM D-1557.
- Structural fill placed against subgrade walls should be compacted to between 90 and 92 percent of the MDD per ASTM D-1557. Care should be taken when compacting fill against subsurface walls to avoid over-compaction and, hence overstressing the walls.

We recommend that GeoEngineers be present during probing of the exposed subgrade soils in building and pavement areas, and during placement of structural fill. We will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to verify compliance with the compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

Weather Considerations

The on-site soils contain a sufficient percentage of fines (silt and clay) to be moisture-sensitive. When the moisture content of these soils is more than a few percent above the optimum moisture content, these soils become muddy and unstable, and operation of equipment on these soils is difficult. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather. During wet weather, we recommend the following:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded such that areas of ponded water do not develop. The contractor should take measures to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.
- Haul roads and staging/laydown areas may need to be constructed with a working pad of quarry spalls if construction is completed during wet weather months (October through May). Typically, a 12- to 18-inch-thick layer of quarry spalls is sufficient for haul roads and staging/laydown areas.

Temporary Slopes

Temporary slopes may be used around the site to facilitate early installation of shoring or in the transition between levels at the base of the excavation. We recommend that temporary slopes constructed in the fill and recent deposits be inclined at 1½H:1V (horizontal to vertical) and that temporary slopes in the glacially consolidated soils be inclined at 1H:1V. Flatter slopes may be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs. For open cuts at the site, we recommend that:

- No traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least 5 feet from the top of the cut;
- Exposed soil along the slope be protected from surface erosion by using waterproof tarps or plastic sheeting;
- Construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the extent practicable;
- Erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practicable;
- Surface water be diverted away from the slope; and
- The general condition of the slopes be observed periodically by the geotechnical engineer to confirm adequate stability.

Because the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. Shoring and temporary slopes must conform to applicable local, state and federal safety regulations.

Recommended Additional Geotechnical Services

GeoEngineers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.

During construction, GeoEngineers should observe the installation of the shoring system, review/collect shoring monitoring data, observe the installation of the ground improvement, evaluate the suitability of the foundation subgrades, observe installation of subsurface drainage measures, evaluate structural backfill, observe the condition of temporary cut slopes, and provide summary letter(s) of our construction observation services. The purposes of GeoEngineers construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix H, Report Limitations and Guidelines for Use.

LIMITATIONS

We have prepared this report for the exclusive use of Willow Run, LLC and their authorized agents for the Building X project in Redmond, Washington.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix H for additional information pertaining to use of this report.

REFERENCES

Aspect Consulting, 2017. "Draft Memorandum, Project Mammoth, Preliminary Environmental and Geotechnical Considerations."

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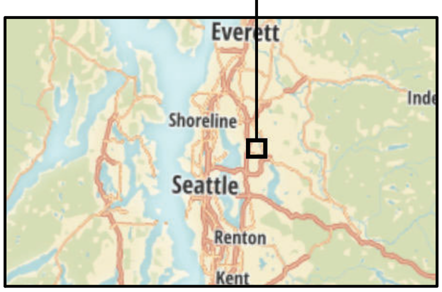
FIGURES



N
W E
S

2,000 0 2,000

Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2017

Projection: NAD 1983 StatePlane Washington South FIPS 4602 Feet

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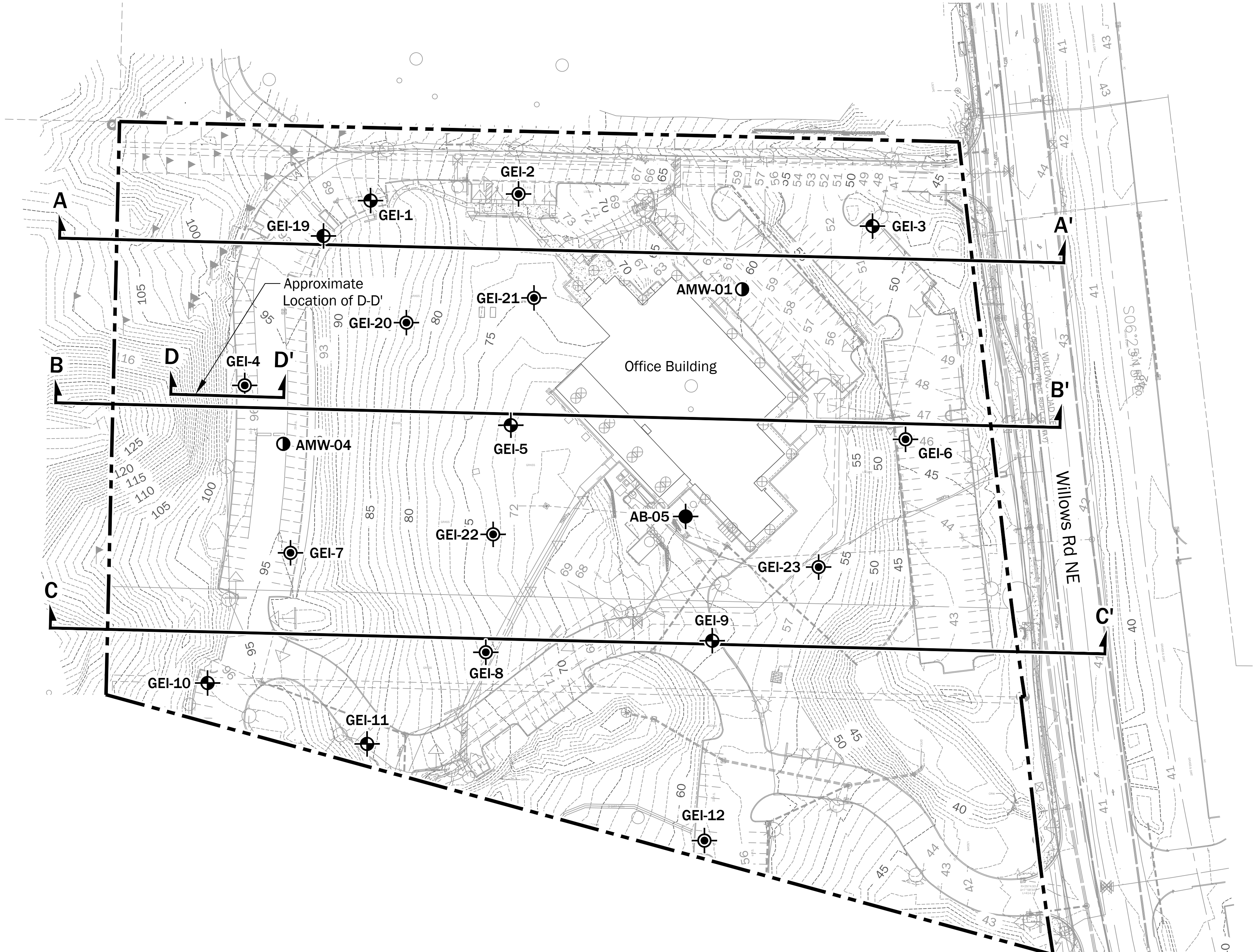
Vicinity Map	
Building X Redmond, Washington	
GEOENGINEERS	Figure 1



Legend

- Property Boundary
- GEI-1 Boring with Monitoring Well by GeoEngineers, 2018
- GEI-2 Boring by GeoEngineers, 2018
- GEI-19 Sonic Core Boring with Test Well by GeoEngineers, 2018
- AMW-01 Boring with Monitoring Well by Aspect Consulting, 2017
- AB-05 Boring by Aspect Consulting, 2017
- A A' Cross Section Location

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Site Survey by Bush Roed and Hitchings dated 01/21/2019. Building Outline by Gehry Partners dated April 12, 2019.

Projection: WA State Plane, North Zone, NAD83, US Foot

N E S W

100 0 100

Feet

Site Plan

Building X
Redmond, Washington

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Figure 2

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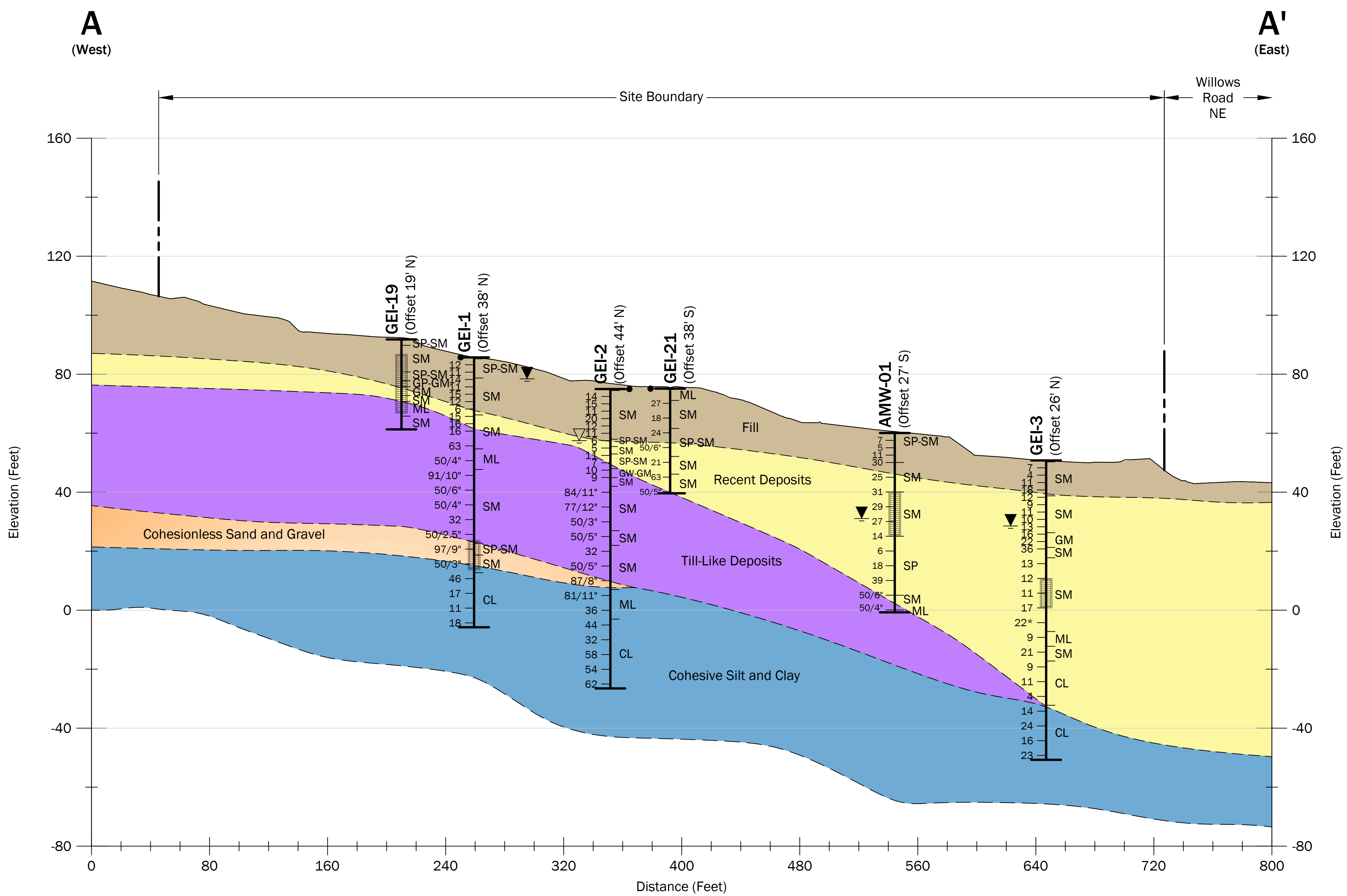

Notes:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Data Source: Site Survey by Bush Roed and Hitchings dated 01/21/2019. Building Outline by Gehry Partners dated April 12, 2019.

Datum: NAVD 88, unless otherwise noted.

Legend

GEI-1 (Offset 12' N) — Boring ID

Inferred Soil Contact

SM — Soil Classification

Water Observed During Drilling

Groundwater Measured in Piezometer

Well Screen

20 — Blow Count

Fill

Recent Deposits

Till-Like Deposits

Cohesionless Sand and Gravel

Cohesive Silt and Clay

80 0 80
Horizontal Scale in Feet

40 0 40
Vertical Scale in Feet
Vertical Exaggeration: 2X

Cross Section A-A'

Building X
Redmond, Washington

GEOENGINEERS

Figure 3

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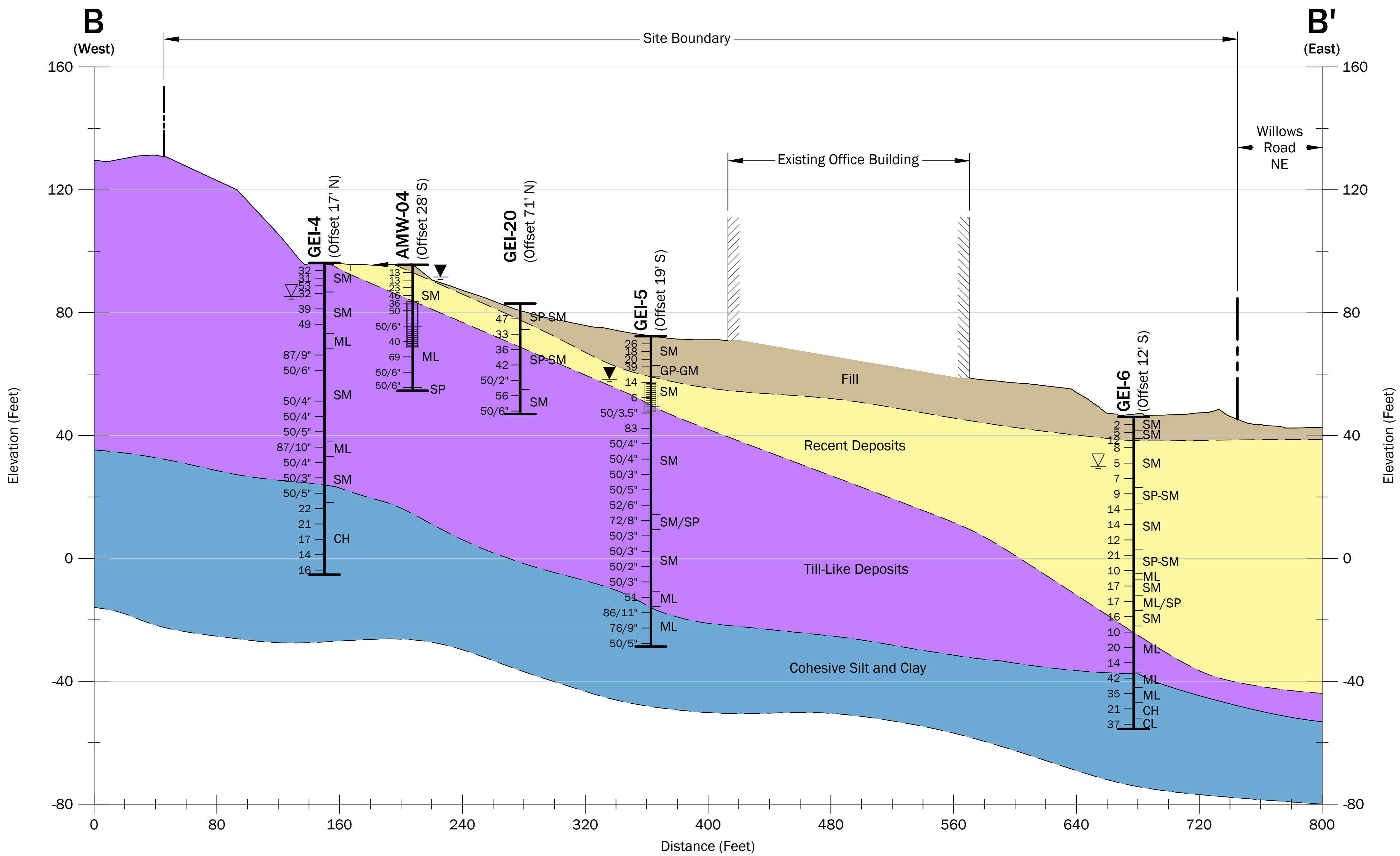

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Data Source: Site Survey by Bush Roed and Hitchings dated 01/21/2019. Building Outline by Gehry Partners dated April 12, 2019.

Datum: NAVD 88, unless otherwise noted.

Legend

GEI-4 (Offset 16' N) — Boring ID
Inferred Soil Contact
SM — Soil Classification
Water Observed During Drilling
Groundwater Measured in Piezometer
Well Screen
20 — Blow Count

Fill
Recent Deposits
Till-Like Deposits
Cohesive Silt and Clay

80 0 80
Horizontal Scale in Feet
40 0 40
Vertical Scale in Feet
Vertical Exaggeration: 2x

Cross Section B-B'

Building X
Redmond, Washington

GEOENGINEERS

Figure 4

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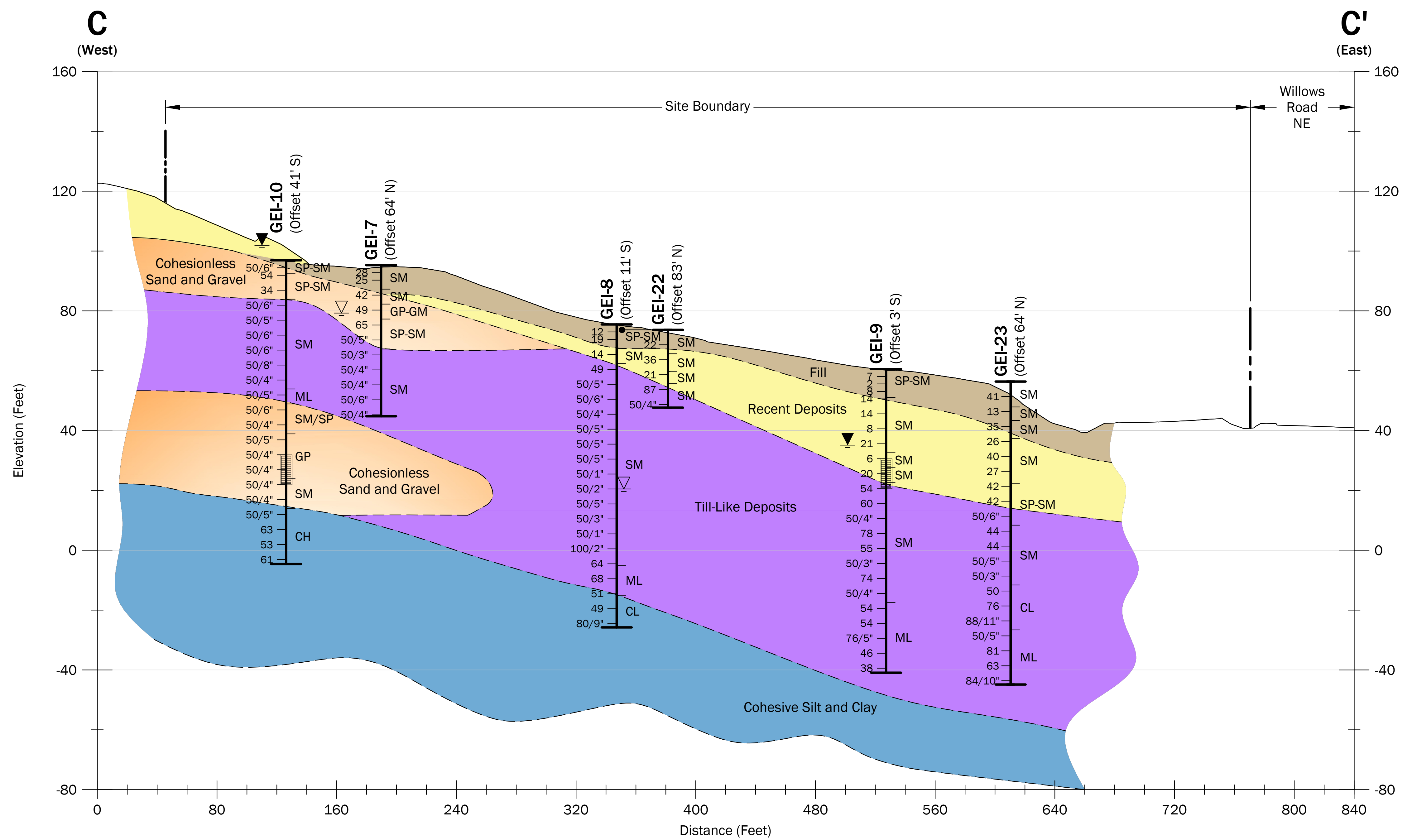

Notes:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Data Source: Site Survey by Bush Roed and Hitchings dated 01/21/2019. Building Outline by Gehry Partners dated April 12, 2019.

Datum: NAVD 88, unless otherwise noted.

Legend

GEI-7 (Offset 72' N) — Boring ID
Inferred Soil Contact
SM — Soil Classification
Water Observed During Drilling
Groundwater Measured in Piezometer
Well Screen
20 — Blow Count

Fill
Recent Deposits
Till-Like Deposits
Cohesionless Sand and Gravel
Cohesive Silt and Clay

80 0 80
Horizontal Scale in Feet

40 0 40
Vertical Scale in Feet
Vertical Exaggeration: 2x

Cross Section C-C'

Building X
Redmond, Washington

GEOENGINEERS

Figure 5

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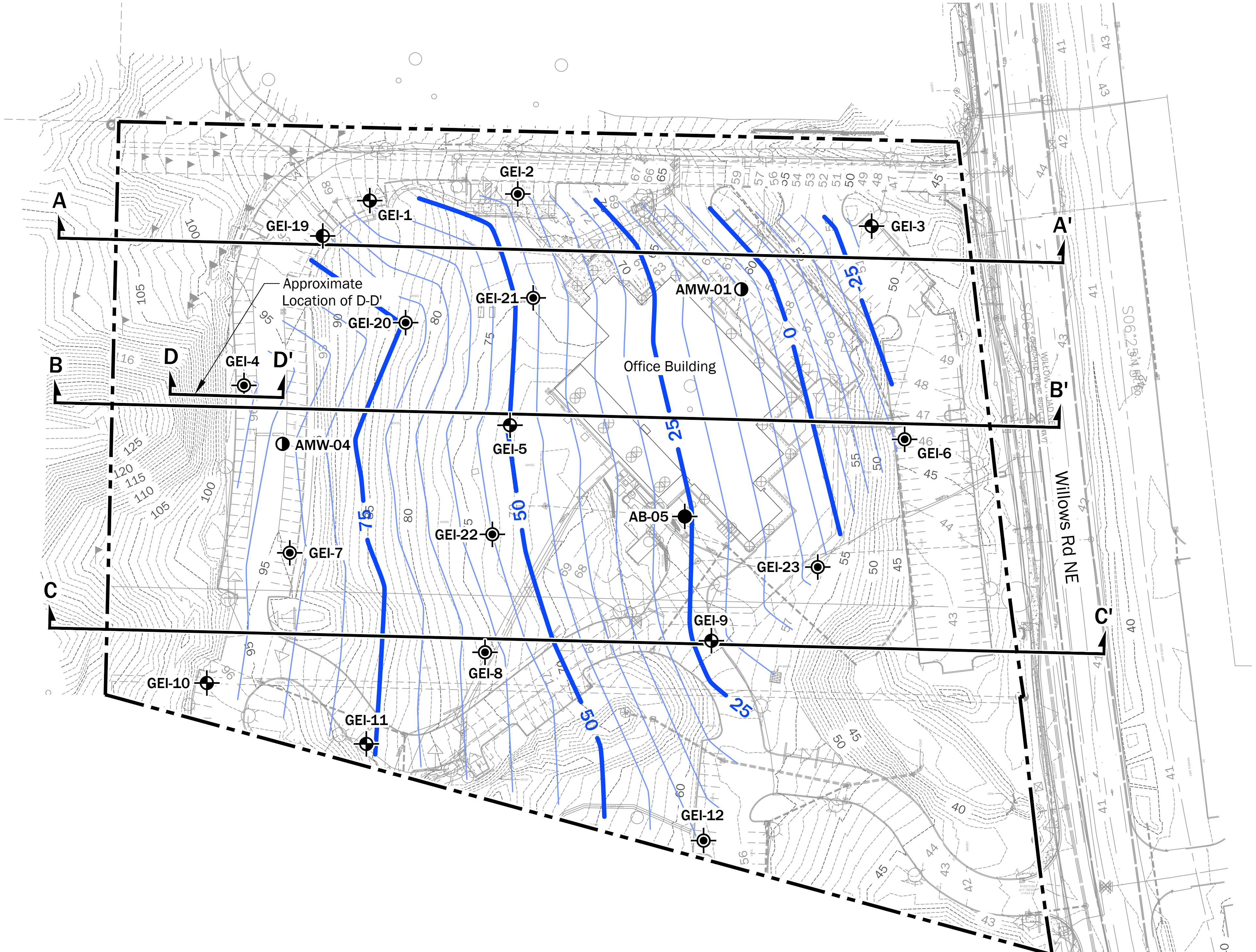

Legend

60 Bearing Soil Contour (feet)

Property Boundary

GEI-1 Boring with Monitoring Well by GeoEngineers, 2018

GEI-2 Boring by GeoEngineers, 2018

GEI-19 Sonic Core Boring with Test Well by GeoEngineers, 2018

AMW-01 Boring with Monitoring Well by Aspect Consulting, 2017

AB-05 Boring by Aspect Consulting, 2017

A A' Cross Section Location

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Site Survey by Bush Roed and Hitchings dated 01/21/2019. Building Outline by Gehry Partners dated April 12, 2019.

Projection: WA State Plane, North Zone, NAD83, US Foot

N
W E
S

100 0 100

Feet

Estimated Top of Bearing Soils

Building X
Redmond, Washington

GEOENGINEERS

Figure 6

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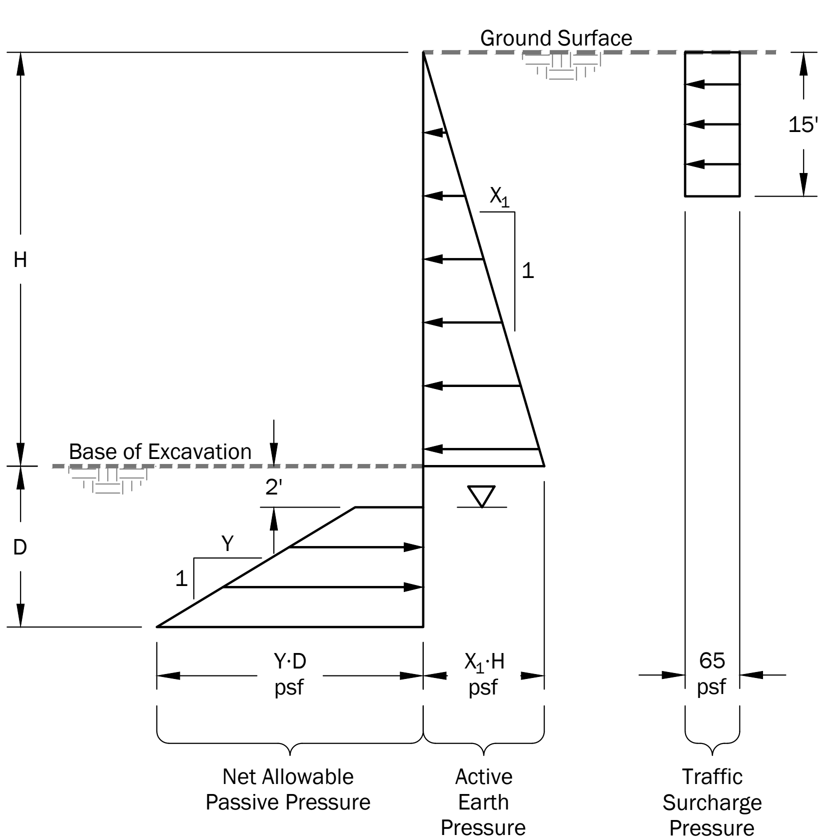
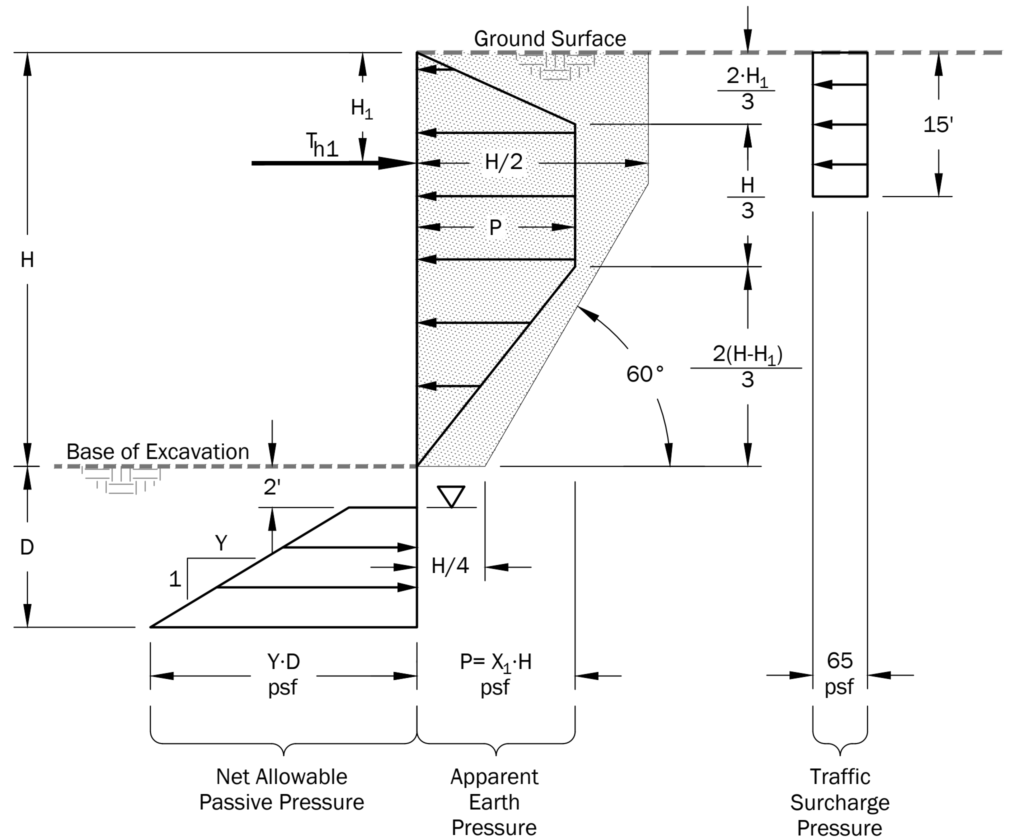
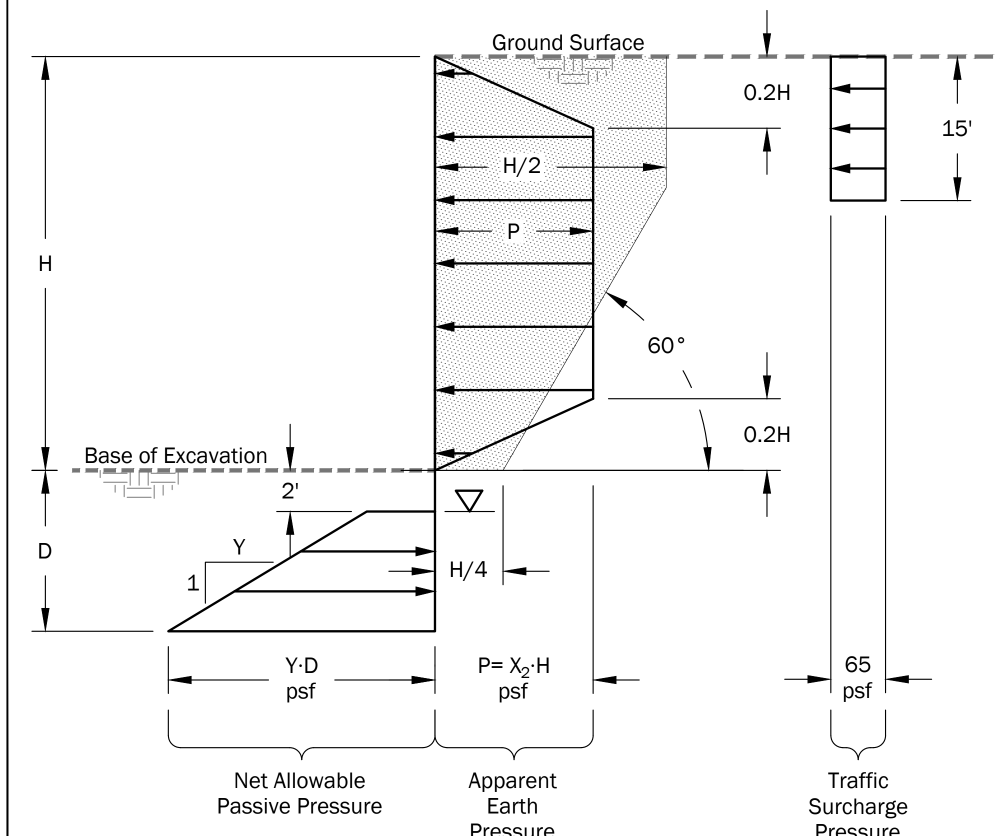
Cantilever Soldier Pile



Conventional Soldier Pile Wall with One Level of Tiebacks



Conventional Soldier Pile Wall with Multiple Levels of Tiebacks



Notes:
1. Active/apparent earth pressure and traffic surcharge pressure act over the pile spacing above the base of the excavation.
2. Passive earth pressure acts over 2.5 times the concreted diameter of the soldier pile, or the pile spacing, whichever is less.
3. Passive pressure includes a factor of safety of 1.5
4. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 8.
5. This pressure diagram is appropriate for temporary soldier pile and tieback walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.
6. Reference table for site specific earth pressure factor.

Soil Unit	Earth Pressure Factor		
	X_1	X_2	Y
Fill and Recent Deposits	31	25	300
Glacially Consolidated Soils	27	22	400

Legend

No Load Zone

H = Height of Excavation, Feet

D = Soldier Pile Embedment Depth, Feet

H_1 = Distance From Ground Surface to Uppermost Tieback, Feet

T_{h1} = Horizontal Load in Uppermost Ground Anchor

P = Maximum Apparent Earth Pressure Pounds per Square Foot

▽ Design Groundwater Elevation for Drained Walls/ Passive Resistance Design

X_1, X_2, Y = Earth Pressure Factor

Not To Scale

Earth Pressure Diagrams - Temporary Soldier Pile & Tieback Wall

Building X
Redmond, Washington

GeoEngineers

Figure 7

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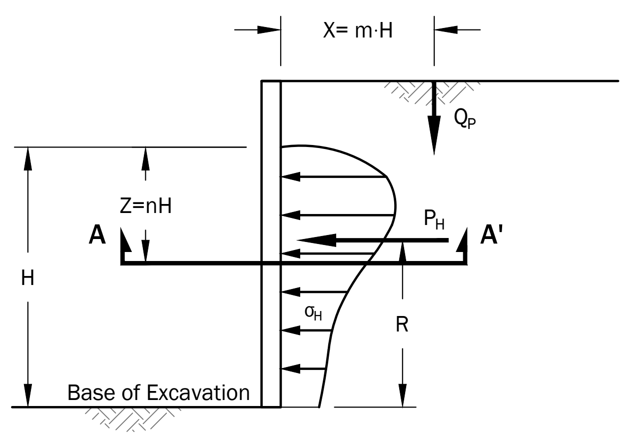
Lateral Earth Pressure from Point Load, Q_P (Spread Footing)



For $m \le 0.4$

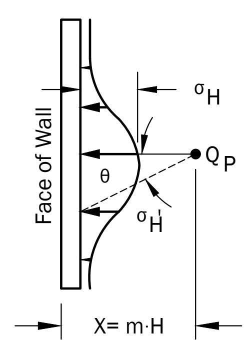
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2(0.16+n^2)^3}$$

For $m > 0.4$

$$\sigma_H = \frac{1.77 Q_P m^2 n^2}{H^2(m^2+n^2)^3}$$

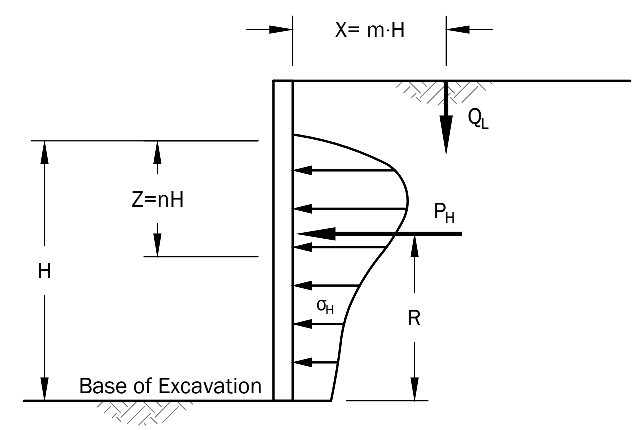
$$\sigma'_H = \sigma \, COS^2 (1.1\theta)$$

m	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



Section A-A'
Pressures from Point Load Q_P

Lateral Earth Pressure from Line Load, Q_L (Continuous Wall Footing)



For $m \le 0.4$

$$\sigma_H = \frac{0.2n \cdot Q_L}{H(0.16+n^2)^2}$$

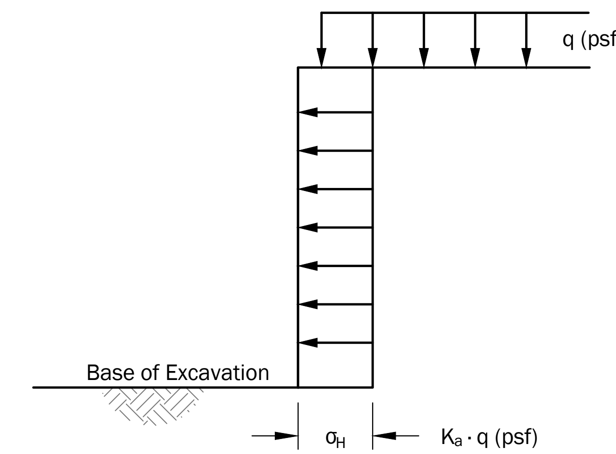
For $m > 0.4$

$$\sigma_H = \frac{1.28 m^2 n Q_L}{H(m^2+n^2)^2}$$

$$\text{Resultant } P_H = \frac{0.64 Q_L}{(m^2+1)}$$

m	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H

Uniform Surcharges, q (Floor Loads, Large Foundation Elements or Traffic Loads)



σ_H = Lateral Surcharge Pressure from Uniform Surcharge

Soil Unit	Earth Pressure Coefficient, K_a
Fill and Recent Deposits	0.25
Glacially Consolidated Soils	0.21

Definitions:

Q_P = Point load in pounds
Q_L = Line load in pounds/foot
H = Excavation height below footing, feet
σ_H = Lateral earth pressure from surcharge, psf
q = Surcharge pressure in psf
θ = Radians
σ'_H = Distribution of σ_H in plan view
P_H = Resultant lateral force acting on wall, pounds
R = Distance from base of excavation to resultant lateral force, feet
X = Resultant lateral force acting on wall, pounds
Z = Depth of σ_H to be evaluated below the bottom of Q_P or Q_L
m = Ratio of X to H
n = Ratio of Z to H

Notes:
1. Procedures for estimating surcharge pressures shown above are based on Manual 7.02 Naval Facilities Engineering Command, September 1986 (NAVFAC DM 7.02).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figures 7.
3. See report text for where surcharge pressures are appropriate.

Recommended Surcharge Pressure

Building X
Redmond, Washington

GeoEngineers

Figure 8

APPENDICES

APPENDIX A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

Subsurface conditions were explored at the site by drilling 17 borings (GEI-1 through GEI-12, and GEI-19 through GEI-23). These borings were completed to depths ranging from approximately 26 and 101½ feet below the existing ground surface (bgs). GEI-1, GEI-3, GEI-5, GEI-9, GEI-10, and GEI-11 were completed with 2-inch diameter monitoring wells. GEI-19 was completed as a large (6-inch) diameter test well. Borings GEI-1 through GEI-6 and GEI-9 were completed by Holt Services Inc. between February 19 through March 8, 2018. Borings GEI-7, GEI-8, and GEI-10 through GEI-12 were completed by Holocene Drilling, Inc. between March 5 and 9, 2018. Borings GEI-19 through GEI-23 were completed by Holocene Drilling, Inc. between November 26 and December 11, 2018. Borings GEI-13 through GEI-18 were completed on a property to the south of the project site and are not included in this report.

The locations of the explorations were estimated by taping/pacing from existing site features. The approximate exploration locations are shown on the Figure 2.

Borings

The borings, except for GEI-19, were completed using track-mounted and truck-mounted, continuous-flight, hollow-stem auger drilling equipment and mud rotary drilling equipment. GEI-19 was completed using a Geoprobe 8410 track-mounted sonic drilling equipment. The borings were continuously monitored by a geotechnical engineer or geologist from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions and prepared a detailed log of each exploration.

The soils encountered in the borings were generally sampled at 2½- and 5-foot vertical intervals with a 2-inch outside diameter split-barrel standard penetration test (SPT) sampler. The disturbed samples were obtained by driving the sampler 18 inches into the soil with a 140-pound hammer free-falling 30 inches. The number of blows required for each 6 inches of penetration was recorded. The blow count ("N-value") of the soil was calculated as the number of blows required for the final 12 inches of penetration. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Where very dense soil conditions precluded driving the full 18 inches, the penetration resistance for the partial penetration was entered on the logs. The blow counts are shown on the boring logs at the respective sample depths. The soils encountered in boring GEI-19 were obtained from the continuous sonic core (i.e. grab samples).

Soils encountered in the borings were visually classified in general accordance with the classification system described in Figure A-1. A key to the boring log symbols is also presented in Figure A-1. The logs of the borings are presented in Figures A-2 through A-18. The boring logs are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change may actually be gradual. If the change occurred between samples, it was interpreted. The densities noted on the boring logs are based on the blow count data obtained in the borings and judgment based on the conditions encountered.

Observations of groundwater conditions were made during drilling. The groundwater conditions encountered during drilling are presented on the boring logs. Groundwater conditions observed during drilling represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate.

Monitoring Wells

A representative of GeoEngineers observed the installation of a monitoring well in borings GEI-1, GEI-3, GEI-5, GEI-9, GEI-10, and GEI-11. The monitoring wells were constructed using 2-inch-diameter polyvinyl chloride (PVC) casing. A test well was constructed in boring GEI-19. The test well was constructed using 6-inch-diameter PVC casing and well screen. The depth to which the casing was installed was selected based on our understanding of subsurface soil and groundwater conditions in the project area. The lower portion of the casing was slotted to allow entry of water into the casing. Medium sand was placed in the borehole annulus surrounding the slotted portion of the casing. A bentonite seal was placed above and below the slotted portion of the casing. The monitoring well was protected by installing a flush-mount steel monuments set in concrete. Completion details for the monitoring wells and test well are shown on the exploration logs presented in Appendix A.

The monitoring wells and test well located within the planned building footprint or excavation area will require decommissioning by a licensed well driller prior to the start of excavation activities. The decommissioning of the wells includes backfilling the monitoring wells and providing documentation of the decommissioning to the Washington State Department of Ecology (Ecology). The well installation logs and Ecology registry information required for decommissioning and documentation is included on the boring logs in Appendix A.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

GEOENGINEERS

Figure A-1

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	2/19/2018	2/21/2018	91.5	SLG	JDB	Holt Services, Inc.	Hollow-stem Auger & Mud Rotary

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile B58	DOE Well I.D.: BKL-414	A 2-in well was installed on 2/21/2018 to a depth of 71.75 ft.
Surface Elevation (ft) / Vertical Datum	85.64 / NAVD88	Top of Casing Elevation (ft)	85.35		
Easting (X) / Northing (Y)	1315342 / 255562	Horizontal Datum	WA State Plane North NAD83 (feet)		

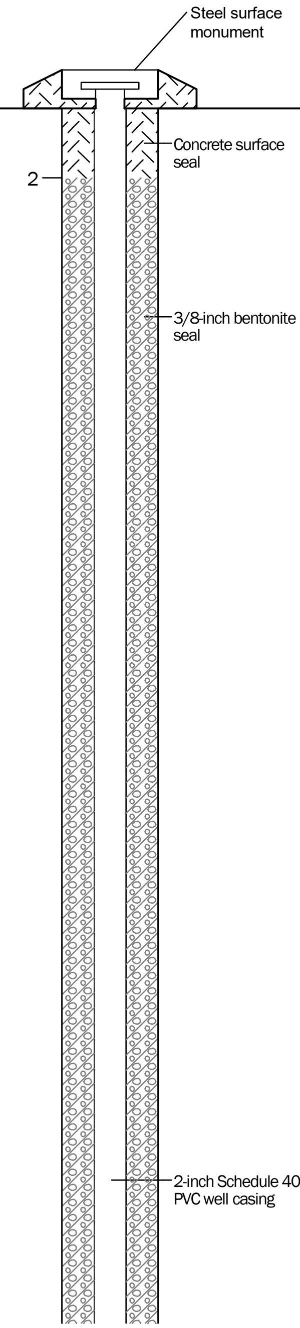
Groundwater

Date Measured	Depth to Water (ft)	Elevation (ft)
3/28/2018	6.70	78.65

Notes:

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
85	0					AC	5 inches asphalt concrete pavement		
						SP-SM	Brown fine to medium sand with gravel and silt (medium dense, moist) (fill)		
		4	12	1					
80	5	2	11	2			Becomes wet (Sample wet, rods dry, sample appears to be all slough)		
					▼				
		4	14	3 / %F		SM	Brown silty fine to medium sand with gravel (loose to medium dense, wet)	9	12
75	10	4	11	4 / %F				8	14
		4	15	5 / %F			(Driller noted heave at approximately 12 feet; added water to auger)	8	13
70	15	4	12	6 / %F				11	26
		6	6	7 / %F			Grades to with occasional gravel (Rod sanded in annulus at 17½ feet; switched to mud rotary)	13	25
65	20	6	15	8 / %F		SM	Gray silty fine to medium sand with gravel (medium dense, wet) (recent deposits)	12	30
		6	16	9					
60	25	6	16	10					
55	30	12	63	11					
				12		ML	Gray sandy silt with gravel (hard, moist) (glacially consolidated soils) (Driller noted difficult drilling to 35 feet)		
	35								

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

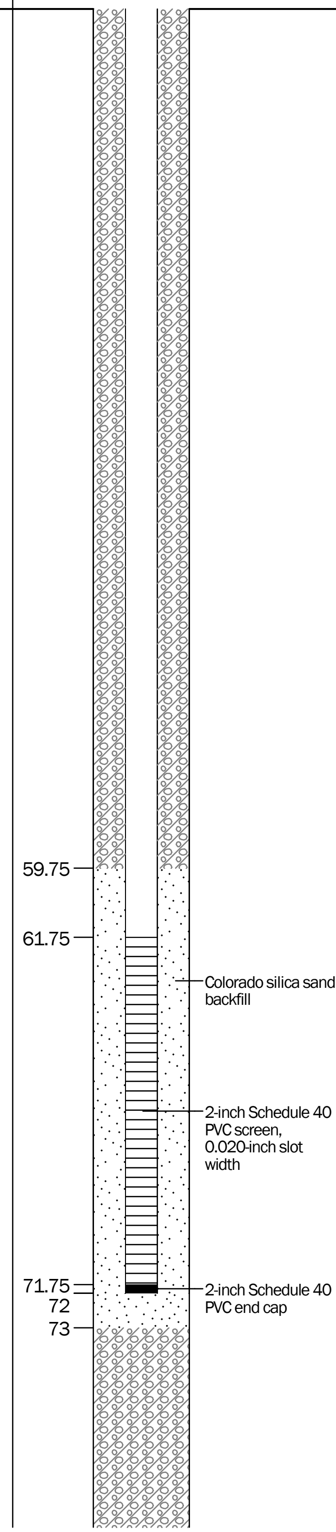
Log of Boring with Monitoring Well GEI-1

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
50	35		10	50/4"		13 %F					14	56
										(Driller noted difficult drilling to 40 feet)		
									SM	Gray silty fine to medium sand with gravel (dense to very dense, moist to wet)		
45	40		16	91/10"		14						
40	45		6	50/6"		15				Becomes moist		
35	50		6	50/4"		16 %F					10	21
30	55		0	32		17				(No recovery)		
25	60		1.5	50/2.5"		18				(Poor recovery)		
										(Driller noted easier drilling at 62 feet)		
									SP-SM	Gray fine to medium sand with silt and silt interbeds (very dense, wet)		
20	65		12	97/9"		19						
									SM	Gray silty fine to medium sand with gravel (very dense, wet)		
15	70		2	50/3"		20 %F					9	30
									CL	Gray clay (stiff to hard, moist to wet)		
10	75		18	46		21						

WELL LOG



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Log of Boring with Monitoring Well GEI-1 (continued)

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-2
Sheet 2 of 3

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	FIELD DATA: Recovered (in)	FIELD DATA: Blows/foot	FIELD DATA: Collected Sample	FIELD DATA: Sample Name / Testing	FIELD DATA: Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
5	80		18	17		22						
										(Clay cuttings collaring off rods during removal)		
0	85		18	11		23						
-5	90		18	18		24						

WELL LOG

91.5

Log of Boring with Monitoring Well GEI-1 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Drilled	Start 2/22/2018	End 2/26/2018	Total Depth (ft) 101.5	Logged By SLG Checked By JDB/MWS	Driller Holt Services, Inc.	Drilling Method Hollow-stem Auger & Mud Rotary

Surface Elevation (ft) 75 Vertical Datum NAVD88	Hammer Data Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment Mobile B58
Easting (X) 1315500 Northing (Y) 255590	System Datum WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	5 inches asphalt concrete pavement			
								SM	Brown silty fine to medium sand with gravel (medium dense, moist) (fill)			
			12	14		1						
70	5		12	15		2						
			0	11		3						No recovery; cuttings indicate same as above
65	10		0	20		4						No recovery; cuttings indicate same as above
			2	12		5						Poor recovery; rock stuck in sampler shoe
60	15		4	11		6						
			4	6		7 / %F		SP-SM	Gray fine to medium sand with silt and gravel (loose, wet) (recent deposits)	8	10	Perched water observed at 17½ feet
55	20		3	5		8 / %F		SM	Gray silty fine to medium sand with gravel (loose, wet)	10	13	
			3	11		9 / %F		SP-SM	Gray fine to medium sand with silt and gravel (loose to medium dense, wet)	8	11	Driller noted 3 feet of heave; added water to augers
50	25		3	7		10 / %F				10	11	
			4	10		11 / SA		GW-GM	Gray fine gravel with silt and sand (loose to medium dense, wet)	8	6	
45	30		6	9		12 / %F / 13		SM	Gray silty fine to medium sand with occasional gravel (loose, moist)	9	12	
								SM	Gray silty fine to medium sand with gravel; till-like (very dense, moist) (glacially consolidated soils)			
40	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring GEI-2

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		17	84/11"		14						
35	40		16	77/12"		15						
30	45		9	50/3"		16						
25	50		5	50/5"		17			Grades to with trace gravel and gray silt interbeds; becomes wet			
20	55		12	32		18			Grades to with gravel			
15	60		5	50/5"		19						
10	65		8	87/8"		20						Driller noted 5 to 6 feet of heave; switched to mud rotary
								ML	Gray sandy silt (hard, moist to wet)			
5	70		12	81/11"		21 %F				16	50	
0	75		18	36		22						

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
								CL	Gray lean clay with sand (very stiff to hard, moist to wet)			
-5	80		18	44		23 / AL				21		AL (LL = 41; PI = 24)
												Driller noted gravels at 82 feet
-10	85		18	32		24						
-15	90		18	58		25						
-20	95		18	54		26						
-25	100		18	62		27						

Log of Boring GEI-2 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-3
Sheet 3 of 3

Drilled: Start 2/26/2018, End 2/27/2018	Total Depth (ft): 101.5	Logged By: SLG; Checked By: JDB	Driller: Holt Services, Inc.
Drilling Method	Hollow-stem Auger & Mud Rotary		
Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile B58
Surface Elevation (ft) / Vertical Datum	50.69 / NAVD88	Top of Casing Elevation (ft)	50.52
Easting (X) / Northing (Y)	1315766 / 255570	Horizontal Datum	WA State Plane North NAD83 (feet)

DOE Well I.D.: BKL-416
A 2-in well was installed on 2/28/2018 to a depth of 49.5 ft.

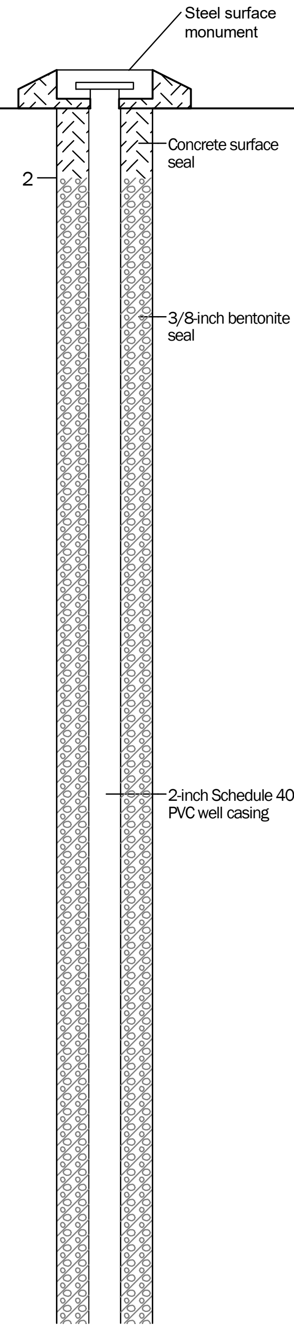
Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
3/28/2018	22.90	27.62

Notes:

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
50	0					AC	5 inches asphalt concrete pavement		
						SM	Brown silty fine to medium sand with gravel (very loose to medium dense, moist) (fill)		
		12	7	1					
45	5	8	4	2					
		8	11	3			Grades to with trace silt		
40	10	0	18	4			(No recovery)		
		10	12	5 %F		SM	Gray silty fine to medium sand with gravel (loose to medium dense, moist) (recent deposits)	13	18
35	15	0	9	6			(No recovery, rock stuck in sampler shoe)		
		6	11	7					
30	20	6	10	8 %F			Grades to with occasional gravel	15	22
		6	13	9	▼		Becomes wet		
25	25	6	16	10 %F		GM	Gray silty fine to coarse gravel with sand (medium dense, wet)	10	15
		0	22	11			(No recovery)		
20	30	12	36	12 SA		SM	Gray silty fine to coarse sand with gravel (dense, wet)	9	17
						SM	Gray silty fine sand with gravel (medium dense, wet)		
	35								

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

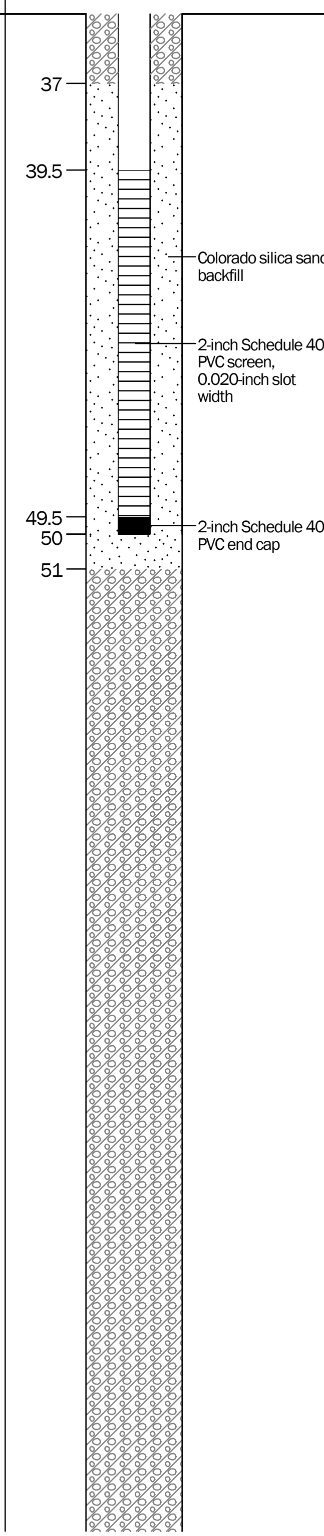
Log of Boring with Monitoring Well GEI-3

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
15	35	8	13	13 %F				16	16
10	40	8	12	14 %F			Grades to with occasional gravel	19	23
5	45	8	11	15 %F			Grades to without gravel	19	18
0	50	10	17	16 %F			Grades to with occasional gravel	20	18
-5	55	0	22*	17			(No recovery, pushing large rock with sampler shoe)		
						ML	Gray silt with sand (stiff, wet)		
-10	60	16	9	18 %F 19				28	73
						SM	Gray silty fine to coarse sand with gravel and gray silt interbeds (medium dense, wet)		
-15	65	8	21	20 %F				29	14
						CL	Gray lean clay with sand, occasional slickensides (soft to stiff, moist to wet) (glacially consolidated soils)		
-20	70	18	9	21					
-25	75	0	11	22			(No recovery)		

WELL LOG



Log of Boring with Monitoring Well GEI-3 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-4
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
-30	80		18	4		23 / AL				AL (LL = 38, PI = 15)	31	
									CL	Gray clay, occasional slickensides (stiff to very stiff, moist to wet)		
-35	85		18	14		24						
-40	90		18	24		25						
-45	95		18	16		26						
-50	100		18	23		27						

WELL LOG

101.5

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

Log of Boring with Monitoring Well GEI-3 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-4
Sheet 3 of 3

Drilled	Start 3/2/2018	End 3/5/2018	Total Depth (ft) 101.5	Logged By SLG Checked By JDB/MWS	Driller Holt Services, Inc.	Drilling Method Hollow-stem Auger & Mud Rotary

Surface Elevation (ft) Vertical Datum	96.21 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile B58
Easting (X) Northing (Y)	1315288 255436	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	4½ inches asphalt concrete pavement			
95								SM	Gray silty fine to medium sand with gravel; till-like (dense to very dense, moist) (glacially consolidated soil)			
			8	32		1						
90	5		0	31		2						No recovery
			4	53		3						Sampler stuck in annulus
85	10		8	32		4 %F		SM	Gray silty fine to coarse sand with occasional gravel (dense, moist)	11	33	Sampler wet Perched water observed at 11 feet during drilling
80	15		10	39		5 %F			Grades to with gravel and silt interbeds; oxidation stains	13	41	
75	20		8	49		6 %F			Becomes wet	11	30	Added water before advancing to 25 feet
								ML	Gray silt with occasional sand (hard, moist)			
70	25		10	54		7						
								SM	Gray silty fine to coarse sand with gravel (very dense, moist)			
65	30		9	87/9"		8 %F				9	29	Switched to mud rotary
	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring GEI-4

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
60	35		12	50/6"		9						
55	40		11	50/5.5"		10 / %F				9	29	
50	45		4	50/4"		11						
45	50		8	50/4"		12						
40	55		9	50/5"		13						
								ML	Gray silt with sand (hard, moist)			
35	60		14	87/10"		14 / %F				24	80	
								SM	Gray silty fine to medium sand with gravel (very dense, wet)			
30	65		4	50/4"		15 / %F				9	28	
25	70		3	50/3"		16						
20	75		5	50/5"		17 / %F			Grades to without gravel	14	36	

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring GEI-4 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
								CH	Gray fat clay (very stiff, moist to wet)			
15	80		18	22		18 / AL				25		AL (LL = 60; PI = 36)
10	85		18	21		19						
5	90		18	17		20 / AL				27		AL (LL = 64; PI = 38)
0	95		18	14		21						
-5	100		18	16		22						

Log of Boring GEI-4 (continued)

GeoEngineers

Project: Building X

Project Location: Redmond, Washington

Project Number: 23237-002-01

Figure A-5

Drilled	Start	End	Total Depth (ft)	Logged By	Driller	Drilling Method
	3/5/2018	3/7/2018	101	JQS; Checked By JDB	Holt Services, Inc.	Hollow-stem Auger & Mud Rotary

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile B57
Surface Elevation (ft) / Vertical Datum	72.26 / NAVD88	Top of Casing Elevation (ft)	71.72
Easting (X) / Northing (Y)	1315464 / 255464	Horizontal Datum	WA State Plane North NAD83 (feet)

DOE Well I.D.: BKL-419
A 2-in well was installed on 3/7/2018 to a depth of 25 ft.

Groundwater

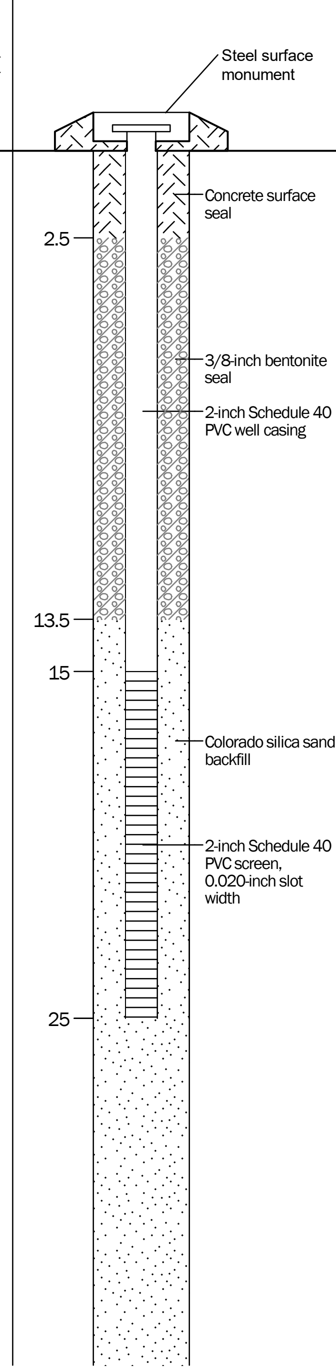
Date Measured	Depth to Water (ft)	Elevation (ft)
3/28/2018	13.80	57.92

Notes: Monitoring well installed 5 feet north

FIELD DATA

Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
0				SOD	3 inches sod and grass		
				SM	Brown silty fine to medium sand with gravel (medium dense, moist) (fill)		
	10	26	1				
5	8	18	2		Becomes moist to wet		
	6	20	3		Becomes wet		
10	8	39	4	GP-GM	Brown fine to coarse gravel with silt and sand (dense, wet)		
				SM	Brown silty fine to medium sand with occasional gravel (loose to medium dense, wet) (recent deposits)		
15	8	14	5 / %F			14	38
20	10	6	6				
				SM	Gray silty fine to medium sand with occasional gravel (very dense, moist) (glacially consolidated soils)		
25	7	50/3.5"	7 / %F			9	29
30	16	83	8				
35							

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Boring with Monitoring Well GEI-5

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-6
Sheet 1 of 3

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	WELL LOG
	35		7	50/4"		9							
35													
	40		9	50/4"		10							
30													
	45		9	50/3"		11							
25													
	50		3	50/5"		12							
20													
	55		6	52/6"		13 %F				Becomes wet	13	33	
15													
									SM/SP	Gray silty fine to medium sand with interbedded fine to medium sand (very dense, moist to wet)			
	60		12	72/8"		14							
10													
									SM	Gray silty fine to coarse sand with gravel (very dense, moist to wet)			
	65		1	50/3"		15							
5													
	70		3	50/3"		16 %F					10	29	
0													
	75		8	50/2"		17							
-5													

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

Log of Boring with Monitoring Well GEI-5 (continued)

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-6
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	WELL LOG
	80		2	50/3"		18							
-10													
									ML	Gray silt with gravel and interbedded layers of fine sand (hard, moist to wet)			
	85		18	51		19							
-15													
	90		17	86/11"		20 %F				Grades to with sand	23	81	
-20													
	95		0	76/9"		21							
-25													
	100		10	50/5"		22							101

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

Log of Boring with Monitoring Well GEI-5 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Drilled	Start 2/28/2018	End 3/1/2018	Total Depth (ft)	101.5	Logged By SLG / Checked By MWS	Driller Holt Services, Inc.	Drilling Method	Hollow-stem Auger & Mud Rotary

Surface Elevation (ft) / Vertical Datum	46.02 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop
Easting (X) / Northing (Y)	1315809 / 255395	System Datum	WA State Plane North NAD83 (feet)

Drilling Equipment: Mobile B58

See "Remarks" section for groundwater observed

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
45	0							AC	1½ inches asphalt concrete pavement			
								SM	Reddish brown silty fine to medium sand with occasional gravel and trace charred wood fragments (very loose, moist) (fill)			
			3	2		1						
40	5		4	5		2		SM	Brown silty fine to medium sand (loose, moist)			
			6	12		3 / %F		SM	Gray brown silty fine to coarse sand (loose to medium dense, moist) (recent deposits)	6	17	
35	10		8	8		4						
30	15		12	5		5 / %F			Becomes wet	17	22	Groundwater observed at 16 feet during drilling
25	20		8	7		6 / %F			Grades to with gravel	13	16	
								SP-SM	Gray brown fine to medium sand with silt (loose, wet)			
20	25		3	9		7 / %F				15	9	Applied water head at 25 feet prior to pulling auger plug
								SM	Gray brown silty fine to coarse sand with gravel (medium dense, wet)			
15	30		8	14		8 / %F				12	13	
	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Boring GEI-6

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
10	35		8	14		9 %F			Grades to with occasional gravel	16	16	Driller noted 18 inch heave; continued adding water to augers
5	40		8	12		10 %F			Grades to without gravel	19	16	Driller noted 18 inch heave
								SP-SM	Gray brown fine to medium sand with silt and occasional gravel (medium dense, wet)			
0	45		4	21		11 %F				13	10	Driller noted 18 inch heave
-5	50		8	10		12						
						13		ML	Gray silt (stiff, wet)			
								SM	Grayish brown silty fine to coarse sand with gravel (medium dense, wet)			
-10	55		6	17		14						
								ML/SP	Gray silt with interbedded layers of fine to coarse sand (very stiff/medium dense, wet)			
-15	60		8	17		15						
								SM	Gray silty fine to coarse sand with gravel (medium dense, wet)			
-20	65		2	16		16 %F				11	14	
								ML	Gray silt with trace sand and occasional gravel (stiff to very stiff, wet) (glacially consolidated soils)			
-25	70		1	10		17						Poor recovery; rock stuck in sampler shoe, silt on outside of sampler
-30	75		16	20		18						

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring GEI-6 (continued)

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
-35	80		16	14		19 / AL				22		AL (LL = 24; PI = 3)
								ML	Gray sandy silt with occasional gravel (hard, wet)			
-40	85		18	42		20						
								ML	Gray silt (hard, moist to wet)			
-45	90		18	35		21						
								CH	Gray fat clay (very stiff, moist)			
-50	95		18	21		22						
								CL	Gray clay (hard, moist to wet)			
-55	100		18	37		23						

Log of Boring GEI-6 (continued)

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Drilled	Start 3/5/2018	End 3/5/2018	Total Depth (ft) 50.5	Logged By CAH / Checked By JDB	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	95.27 / NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: Diedrich (D-120) truck-mounted drill rig
Easting (X) / Northing (Y)	1315326 / 255312	System Datum: WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
95	0							AC	6 inches of asphalt concrete pavement			
								SM	Brown silty fine to medium sand with gravel, trace organic matter (medium dense, moist) (fill)			
			2.5	28		1						
90	5		14	25		2						
								SM	Gray silty fine to coarse sand with gravel (dense, moist) (glacially consolidated soil)			
85	10		10.5	42		3						
								GP-GM	Gray fine to coarse gravel with silt and sand (dense, wet)			
80	15		11	49		4 / SA				9	9	
												Groundwater observed at 16 feet during drilling
								SP-SM	Gray fine to coarse sand with gravel and silt (very dense, wet)			
75	20		14	65		5 / %F				10	11	
70	25		5.5	50/5"		6						
								SM	Gray fine to coarse sand with gravel; till-like (very dense, wet)			
65	30		7	50/3"		7						
	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

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Log of Boring GEI-7

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
60	35		16	50/4"		8 %F			Grades to with seams of fine to medium sand	13	15	
55	40		14	50/4"		9						
50	45		7	50/6"		10						
45	50		4	50/4"		11			Becomes moist to wet			

Log of Boring GEI-7 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-8
Sheet 2 of 2

Drilled	Start 3/9/2018	End 3/9/2018	Total Depth (ft) 101.25	Logged By CAH/JQS, Checked By JDB	Driller Holocene Drilling, Inc.	Drilling Method Mud Rotary

Surface Elevation (ft) / Vertical Datum	75.46 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 850
Easting (X) / Northing (Y)	1315460 / 255322	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Recovered (in)	FIELD DATA: Blows/foot	FIELD DATA: Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
75	0				SP-SM	Brown fine to medium sand with silt and gravel (medium dense, wet) (fill)			
		12	12	1		Grades to with oxidation stains			
70	5	11.5	19	2					
					SM	Brown silty fine to medium sand with gravel (medium dense, moist) (recent deposits)			
65	10	0	14	3					No recovery Soil description based on cuttings
					SM	Gray silty fine to medium sand with gravel; till-like (dense to very dense, moist to wet) (glacially consolidated soils)			
60	15	0	49	4					No recovery
55	20	3.5	50/5"	5 %F			12	30	
50	25	6	50/6"	6					
45	30	0	50/4"	7					No recovery Hard drilling and grinding
	35								

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring GEI-8

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
40	35		7	50/5"		8			Grades to with fine sand interbeds			
35	40		8	50/5"		9						Hard drilling
30	45		2	50/5"		10						
25	50		2	50/1"		11						
									Grades to without fine sand interbeds			
20	55		5	50/2"		12			Becomes moist to wet			
15	60		3	50/5"		13 %F			Becomes wet	15	32	Driller noted possible artesian conditions between 55 and 60 feet
												Driller noted gravels at 64 feet
10	65		2	50/3"		14						
5	70		0.5	50/1"		15						
0	75		7.5	100/2"		16						

Log of Boring GEI-8 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
-5	80		17	64		17		ML	Gray sandy silt (hard, moist to wet)			
-10	85		18	68		18 / %F				16	65	
-15	90		18	51		19		CL	Gray lean clay with interbedded layers of fine sand (hard, wet)			
-20	95		18	49		20			Becomes moist to wet			
-25	100		15	80/9"		21 / AL				30		AL (LL=36; PI=15)

Log of Boring GEI-8 (continued)

GeoEngineers

Project: Building X

Project Location: Redmond, Washington

Project Number: 23237-002-01

Figure A-9

Sheet 3 of 3

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	3/6/2018	3/8/2018	101.5	SLG	JDB	Holt Services, Inc.	Hollow-stem Auger & Mud Rotary

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile B58
Surface Elevation (ft) / Vertical Datum	60.55 / NAVD88	Top of Casing Elevation (ft)	60.26
Easting (X) / Northing (Y)	1315655 / 255230	Horizontal Datum	WA State Plane North NAD83 (feet)

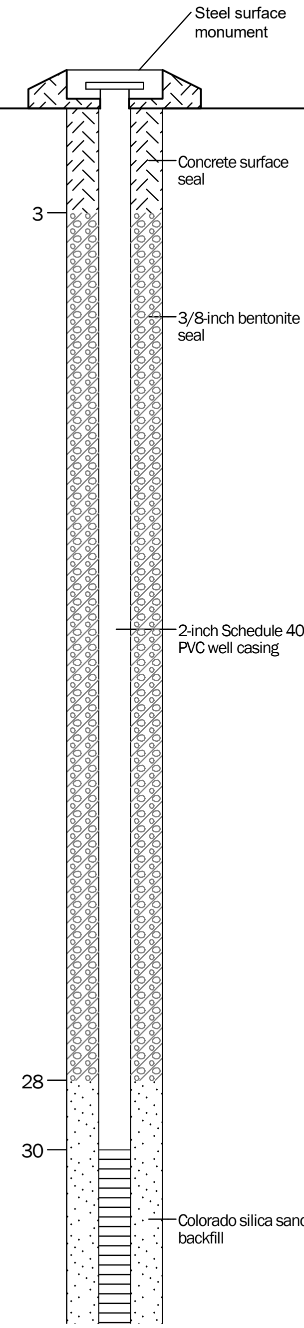
DOE Well I.D.: BKL-415
A 2-in well was installed on 3/8/2018 to a depth of 40 ft.

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
3/28/2018	26.60	33.66

Notes:

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
60	0				AC	5 inches asphalt concrete pavement		
					SP-SM	Brown fine to medium sand with silt and gravel (very loose to loose, moist) (fill)		
		0	7	1		(No recovery)		
55	5	4	2	2				
		4	8	3				
					SM	Gray silty fine to medium sand with occasional gravel (loose to medium dense, moist) (recent deposits)		
50	10	6	14	4				
45	15	8	14	5 %F			4	12
40	20	8	8	6				
35	25	3	21	7				
30	30	8	6	8 %F		(Groundwater observed at 30 feet; switched to mud rotary)	12	37
					SM	Gray silty fine to coarse sand with gravel (medium dense, moist to wet)		
	35							

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Boring with Monitoring Well GEI-9

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-10
Sheet 1 of 3

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	WELL LOG
25	35		8	20		9							2-inch Schedule 40 PVC screen, 0.020-inch slot width
									SM	Gray silty fine to coarse sand with gravel (very dense, moist to wet) (glacially consolidated soil)			
20	40		12	54		10 / %F					14	45	40 / 40.25 — 2-inch Schedule 40 PVC end cap
													43
15	45		18	60		11							
10	50		4	50/4"		12							
5	55		18	78		13							
0	60		18	55		14							
-5	65		2	50/3"		15							
-10	70		18	74		16							3/8-inch bentonite seal
-15	75		4	50/4"		17							
									ML	Gray silt with sand (hard, moist to wet)			

Log of Boring with Monitoring Well GEI-9 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-10
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	WELL LOG
-20	80		18	54		18 / %F					23	78	
-25	85		18	54		19							
-30	90		17	76/5"		20							
-35	95		18	46		21				Grades to with occasional sand			
-40	100		18	38		22 / %F					24	87	101.5

Log of Boring with Monitoring Well GEI-9 (continued)

GEOENGINEERS

Project: Building X

Project Location: Redmond, Washington

Project Number: 23237-002-01

Drilled	Start 3/7/2018	End 3/7/2018	Total Depth (ft)	101.5	Logged By CAH / Checked By JDB	Driller	Holocene Drilling, Inc.	Drilling Method	Hollow-stem Auger & Mud Rotary

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-120 Truck-mounted	DOE Well I.D.: BJL-596 A 2-in well was installed on 3/7/2018 to a depth of 75 ft.
Surface Elevation (ft) / Vertical Datum	96.85 / NAVD88	Top of Casing Elevation (ft)	96.61	
Easting (X) / Northing (Y)	1315255 / 255196	Horizontal Datum	WA State Plane North NAD83 (feet)	

Groundwater

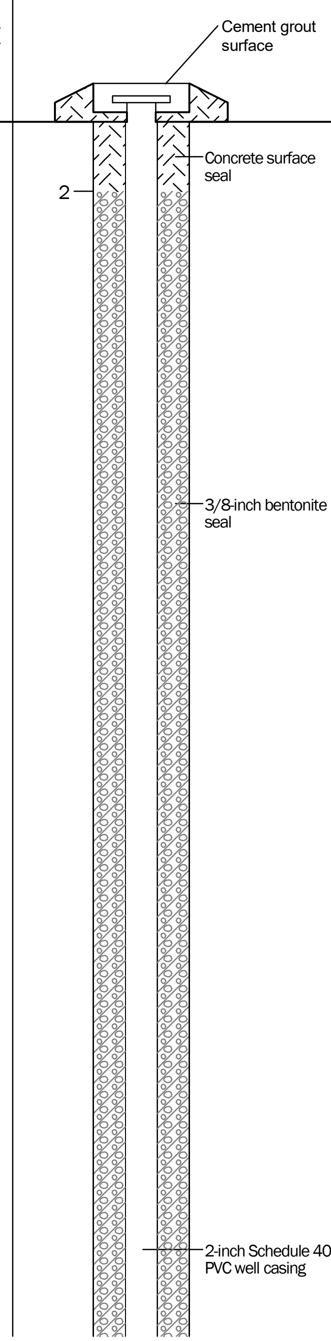
Date Measured	Depth to Water (ft)	Elevation (ft)
3/28/2018	0.00	96.61

Notes: Artesian water conditions

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0				AC	6 inches asphalt concrete pavement		
95					SP-SM	Brown fine to medium sand with silt and sand (fill) (Driller noted hard drilling) (No recovery; descriptions based on cuttings)		
		0	50/6"	1				
	5	8	54	2 / %F	SP-SM	Gray fine to medium sand with gravel and silt (dense to very dense, moist to wet) (glacially consolidated soil) (Driller noted hard drilling)	7	10
90								
	10	6.5	34	3				
85					SM	Gray silty fine to medium sand with gravel (very dense, wet)		
	15	12	50/6"	4		(Groundwater observed at 16 feet)		
80								
	20	7	50/5"	5		Becomes moist		
75								
	25	8.5	50/6"	6 / %F		Grades to with occasional gravel; becomes wet	11	27
70								
	30	6	50/6"	7				
65								
	35							

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Boring with Monitoring Well GEI-10

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	WELL LOG
	35		6	50/8"		8				Becomes moist			
60													
	40		8.5	50/4"		9				Interlayered with fine to medium sand; becomes wet			
55													Colorado silica sand backfill
									ML	Gray silt with sand (hard, moist)			
	45		4.5	50/5"		10 / %F					17	74	
50													
									SM/SP	Gray fine to medium sand and gravel with silty sand interbeds (very dense, wet)			
	50		18	50/6"		11							
45													
	55		9.5	50/4"		12				(Switched to mud rotary)			
40													
									GP	Gray fine gravel (very dense, wet) (*Non-representative samples - suspect soil cuttings lost in mud)			
	60		4	50/5"		13							
35													61
	65		4	50/4"		14							65
30													
	70		3	50/4"		15							2-inch Schedule 40 PVC screen, 0.080-inch slot width
25													
									SM	Gray silty fine sand (very dense, wet)			
	75		2	50/4"		16 / %F					15	44	75
20													77

Log of Boring with Monitoring Well GEI-10 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-11
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	WELL LOG
	80		3.5	50/4"		17							
15													
									CH	Gray fat clay (hard, moist to wet)			
	85		11	50/5"		18							
10													
	90		18	63		19							3/8-inch bentonite seal
5													
	95		18	53		20							
0													
	100		18	61		21							
													101.5

Log of Boring with Monitoring Well GEI-10 (continued)

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

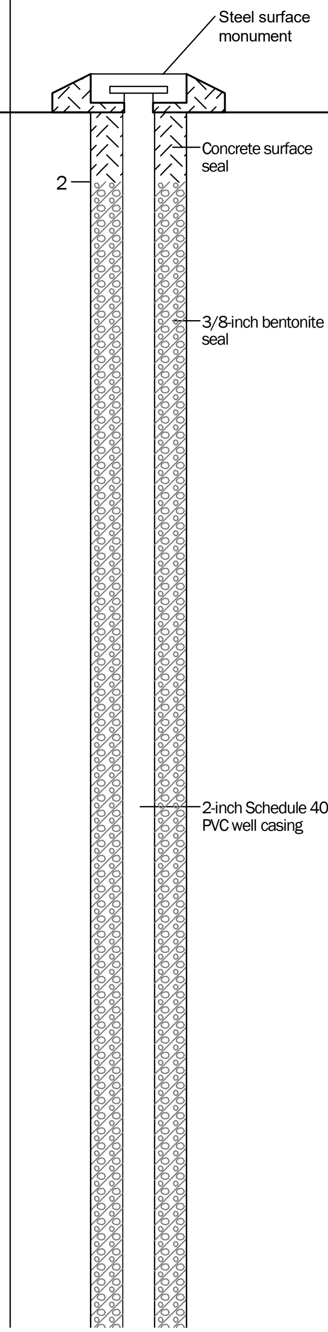
Drilled	Start 3/6/2018	End 3/6/2018	Total Depth (ft) 50.5	Logged By CAH / Checked By JDB	Driller: Holocene Drilling, Inc.	Drilling Method: Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-120 Truck-mounted	DOE Well I.D.: BKC-362 A 2-in well was installed on 3/6/2018 to a depth of 50 ft.
Surface Elevation (ft) Vertical Datum	85.69 NAVD88	Top of Casing Elevation (ft)	85.13	Groundwater
Easting (X) Northing (Y)	1315424 255137	Horizontal Datum	WA State Plane North NAD83 (feet)	Date Measured: 3/28/2018 · Depth to Water (ft): -0.25 · Elevation (ft): 85.38

Notes: Artesian water conditions

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
85	0				AC	2½ inches asphalt concrete pavement		
					CR	2 inches crushed gravel base course		
					GP	Dark gray gravel with sand (very dense, moist) (fill)		
		3	50/6"*	1		(*Blow count not representative; rock stuck in sampler shoe)		
						(Driller noted rough drilling at 4 feet)		
80	5				SM	Brown silty fine to medium sand with gravel (dense, wet)		
		2	44	2				
75	10				SM	Gray silty fine to medium sand with gravel, till-like (very dense, moist to wet) (glacially consolidated soils)		
		12	50/6"	3				
						(Driller noted water at 14 feet)		
70	15							
		12	95	4 %F		Becomes wet	7	17
65	20							
		7.5	75	5 %F		Grades to with thin fine to medium sand seams	12	23
60	25							
		6	50/4"	6				
55	30							
		6	50/5"	7				
	35							

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

Log of Boring with Monitoring Well GEI-11

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
50	35											
			2	50/2"		8 / %F					16	25
45	40											
			4.5	50/6"		9 / %F					10	30
40	45											
			4	50/3"		10 / %F				Becomes moist to wet		
	50		5.5	50/4"		11						

WELL LOG

38
40
Colorado silica sand backfill
2-inch Schedule 40 PVC screen, 0.020-inch slot width
50
50.5
2-inch Schedule 40 PVC end cap

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

Log of Boring with Monitoring Well GEI-11 (continued)

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-12
Sheet 2 of 2

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	3/7/2018	3/7/2018	51.5	CAH	JDB	Holocene Drilling, Inc.	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	57.22 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-120 Truck-mounted
Easting (X) / Northing (Y)	1315649 / 255073	System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0			1	AC	3½ inches asphalt concrete pavement			
55					SP-SM	Brown fine to medium sand with silt and gravel (loose to medium dense, moist) (fill)			
		14	21	2					
	5	5.5	10	3					
50					SM	Gray silty fine to medium sand with gravel (medium dense, moist) (recent deposits)			
	10	12	23	4					
45									
	15	7.5	17	5 %F		Becomes moist to wet	14	24	
40					SP/SM	Gray fine to coarse sand with gravel interbedded with silty fine to medium sand (dense, moist) (glacially consolidated soil)			
	20	7.5	42	6					
35									
	25	12.5	37	7					
30					SM	Gray silty fine sand with gravel; till-like (very dense, moist)			
	30	10	84/11"	8 %F			11	42	
25									
	35								

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Boring GEI-12

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-13
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		0	50/3" 50/2"		9 10			Grades to fine to medium; becomes moist to wet			No recovery Went back down immediately with California sampler; 3-inch, outside-diameter split barrel
20												
	40		4	50/1"		11 %F				6	20	
15												
	45		3.5	50/2"		12			Becomes moist			
10												
	50		5	50/5"		13						

Log of Boring GEI-12 (continued)

GeoEngineers

Project: Building X

Project Location: Redmond, Washington

Project Number: 23237-002-01

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	11/26/2018	11/26/2018	30.5	WCW	MAG	Holocene Drilling, Inc.	Sonic

Hammer Data	N/A	Drilling Equipment	Geoprobe 8140 LC
Surface Elevation (ft) Vertical Datum	91.74 NAVD88	Top of Casing Elevation (ft)	91.15
Easting (X) Northing (Y)	1315344.74 255556.84	Horizontal Datum	WA State Plane North NAD83 (feet)

A 6-in well was installed on 11/26/2018 to a depth of 25 ft.

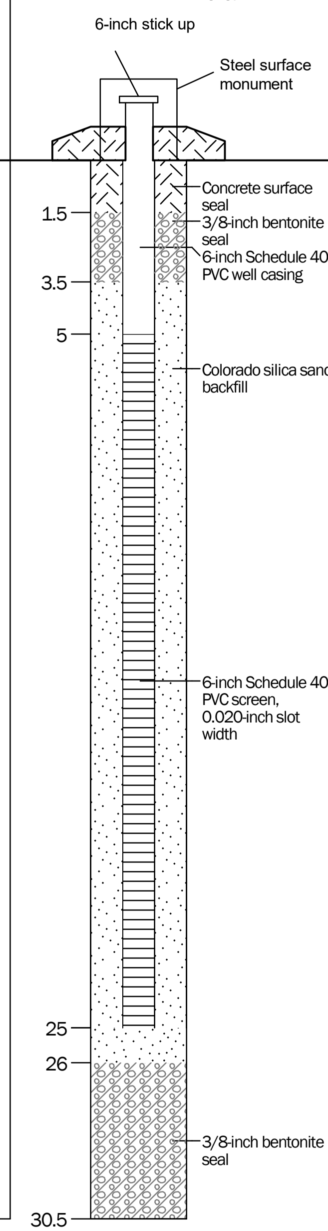
Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
1/22/2019	6.86	84.29

Notes:

FIELD DATA

Elevation (feet)	Depth (feet)	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0			AC	8 inches asphalt concrete pavement		
90		1		SP-SM	Dark brown fine to coarse sand with gravel and silt; base course (medium dense, moist) (fill)		
		2		SM	Brown silty fine to medium sand with gravel, occasional cobbles and trace roots (medium dense, moist)		
	5	3					
85		4 %F	▼		Becomes gray, grades to without roots	3	11
	10	5 %F				4	13
80		6 SA		SP-SM	Brown fine to coarse sand with silt and gravel (medium dense, wet) (recent deposits)	10	9
	15	7 SA		GP-GM	Brown fine to coarse gravel with silt and sand (medium dense, wet)	8	11
75		8 SA		GM	Brown silty fine to coarse gravel with sand (medium dense, wet)	8	13
	20	9 SA		SM	Brown silty fine to medium sand with gravel (medium dense, wet)	10	23
70		10 %F		ML	Gray sandy silt with occasional gravel; till-like (very stiff, moist to wet) (glacially consolidated soils)	16	57
	25	11					
65		12 %F		SM	Gray silty fine to medium sand with gravel; till-like (hard, moist)	10	33
	30	13					

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Locational Survey. Vertical approximated based on Locational Survey.

Log of Boring with Monitoring Well GEI-19

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Drilled	Start 12/11/2018	End 12/11/2018	Total Depth (ft) 36	Logged By SLG / Checked By MAG	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 83 / Vertical Datum NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: CME 850
Easting (X) 1315410.76 / Northing (Y) 255487.8	System Datum: WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SOD	Lawn/grass			
								SP-SM	Gray fine to medium sand with silt and gravel (dense, moist) (recent deposits)			
80												
	5		8	47		1						
75								SP-SM	Gray fine to coarse sand with silt and gravel (dense to very dense, wet) (glacially consolidated soils)			
	10		4	33		2 %F				4	9	Groundwater observed at approximately 10 feet below ground surface during drilling
70												
	15		1	36		3						Poor recovery; rock stuck in sampler shoe
65												
	20		4	42		4 %F				8	10	
60												
	25		0	50/2"		5						No recovery; bouncing on rock
55								SM	Gray silty fine to coarse sand with gravel; till-like (very dense, moist to wet)			
	30		12	56		6 %F				13	35	
50												
	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Locational Survey. Vertical approximated based on Locational Survey.

Log of Boring GEI-20

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-15
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		6	50/6"		7						

Log of Boring GEI-20 (continued)

GeoEngineers

Project: Building X

Project Location: Redmond, Washington

Project Number: 23237-002-01

Drilled	Start 12/11/2018	End 12/11/2018	Total Depth (ft)	35.5	Logged By SLG / Checked By MAG	Driller	Holocene Drilling, Inc.	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) Vertical Datum	75.12 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 850
Easting (X) Northing (Y)	1315512.36 255507.51	System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				SOD	Grass			
					ML	Dark brown sandy silt with trace gravel and trace organic matter (roots) (very stiff, moist) (fill)			
70	5	8	27	1 / %F	SM	Dark brown silty fine to coarse sand with gravel (medium dense moist) (recent deposits)	12	16	
65	10	6	18	2 / %F			5	15	
60	15	8	24	3 / %F	SP-SM	Brown fine to coarse sand with silt and trace gravel (medium dense, moist)	9	10	
						Grades to with occasional gravel			More difficult drilling at 18 feet
55	20	2	50/6"	4					
50	25	2	21	5 / %F	SM	Gray silty fine to coarse sand with gravel (medium dense, moist)	4	14	
45	30	8	63	6	SM	Gray silty fine to medium sand with gravel (very dense, moist) (glacially consolidated soils)			
	35								

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Locational Survey. Vertical approximated based on Locational Survey.

Log of Boring GEI-21

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-16
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		1	50/5"		7			Becomes moist to wet			

Log of Boring GEI-21 (continued)

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-16
Sheet 2 of 2

Drilled	Start 12/11/2018	End 12/11/2018	Total Depth (ft)	26	Logged By SLG / Checked By MAG	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	73.66 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 850
Easting (X) / Northing (Y)	1315479.83 / 255319.24	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	FIELD DATA: Recovered (in)	FIELD DATA: Blows/foot	FIELD DATA: Collected Sample	FIELD DATA: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SOD	Grass			
								SM	Brown silty fine to medium sand with gravel (medium dense, moist) (fill)			
70												
	5		1	22		1						Poor recovery; driving rock
65								SM	Brown silty fine to medium sand with gravel (dense, moist) (recent deposits)			
	10		18	36		2						Groundwater observed at approximately 10 feet below ground surface during drilling
												More difficult drilling at 12 feet
60								SM	Brown silty fine to coarse sand with gravel (medium dense, moist to wet)			
	15		8	21		3 %F				13	34	
55								SM	Gray silty fine to coarse sand with gravel (very dense, moist) (glacially consolidated soils)			
	20		12	87		4 %F				12	30	
50												
	25		10	50/4"		5						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Locational Survey. Vertical approximated based on Locational Survey.

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring GEI-22

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Drilled	Start 12/10/2018	End 12/10/2019	Total Depth (ft) 101.25	Logged By SLG, Checked By MAG	Driller Holocene Drilling, Inc.	Drilling Method Mud Rotary

Surface Elevation (ft) 56.37; Vertical Datum NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: CME 850	
Easting (X) 1315739.02; Northing (Y) 255293.18	System Datum: WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SOD	Grass			
55								SM	Gray silty fine to medium sand with gravel (dense, moist to wet) (fill)			
	5		3	41		1						Blow count not representative
50								SM	Brown silty fine to coarse sand with gravel and charcoal (medium dense, wet)			
	10		12	13		2 %F				21	20	
45								SM	Brown silty fine to medium sand with occasional gravel (dense, wet) (recent deposits)			
	15		6	35		3						
40								SM	Gray silty fine to coarse sand with gravel (medium dense to dense, wet)			
	20		6	26		4 %F				12	23	
35												
	25		6	40		5						
30												
	30		3	27		6 %F				8	15	
25								SP-SM	Gray fine to coarse sand with silt and gravel (dense to very dense, wet) (cohesionless sand and gravel)			
	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Locational Survey. Vertical approximated based on Locational Survey.

Log of Boring GEI-23

GeoEngineers

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Date:2/5/19 Path:P:\23\23237002\GINT\2323700201.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
20	35	8	42	7					
15	40	12	42	8 %F			13	10	
									Less gravel at 48 feet
10	45	0	50/6"	9					No recovery
					SM	Gray silty fine to coarse sand with gravel (dense to very dense, wet) (glacially consolidated soils)			
5	50	12	44	10 %F			12	20	
0	55	12	44	11		Becomes moist to wet; till-like			
-5	60	0	50/5"	12					No recovery
-10	65	0	50/3"	13					No recovery
					CL	Gray clay with trace sand and gravel (hard, moist) (cohesive silt and clay)			
-15	70	18	50	14					
-20	75	18	76	15					

Log of Boring GEI-23 (continued)

GEOENGINEERS

Project: Building X

Project Location: Redmond, Washington

Project Number: 23237-002-01

Figure A-18
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
-25	80		17	88/11"		16			Grades to with sand, moist			
								ML	Gray silt with sand (hard, moist)			
-30	85		11	50/5"		17 / %F				20	79	
-35	90		18	81		18						
-40	95		18	63		19						
	100		14	84/10"		20						

Log of Boring GEI-23 (continued)

GEOENGINEERS

Project: Building X
Project Location: Redmond, Washington
Project Number: 23237-002-01

Figure A-18
Sheet 3 of 3

APPENDIX B
Laboratory Testing

APPENDIX B
LABORATORY TESTING

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of moisture content, percent fines determination, grain size distribution (sieve analyses), and Atterberg Limits testing. The tests were performed in general accordance with test methods of ASTM International (ASTM) or other applicable procedures.

Moisture Content Testing (MC)

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs in Appendix A at the depths at which the samples were obtained.

Percent Passing U.S. No. 200 Sieve (%F)

Selected samples were "washed" through the U.S. No. 200 mesh sieve to determine the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to determine the fines content for analysis purposes. The tests were conducted in general accordance with ASTM D 1140, and the results are shown on the exploration logs in Appendix A at the respective sample depths.

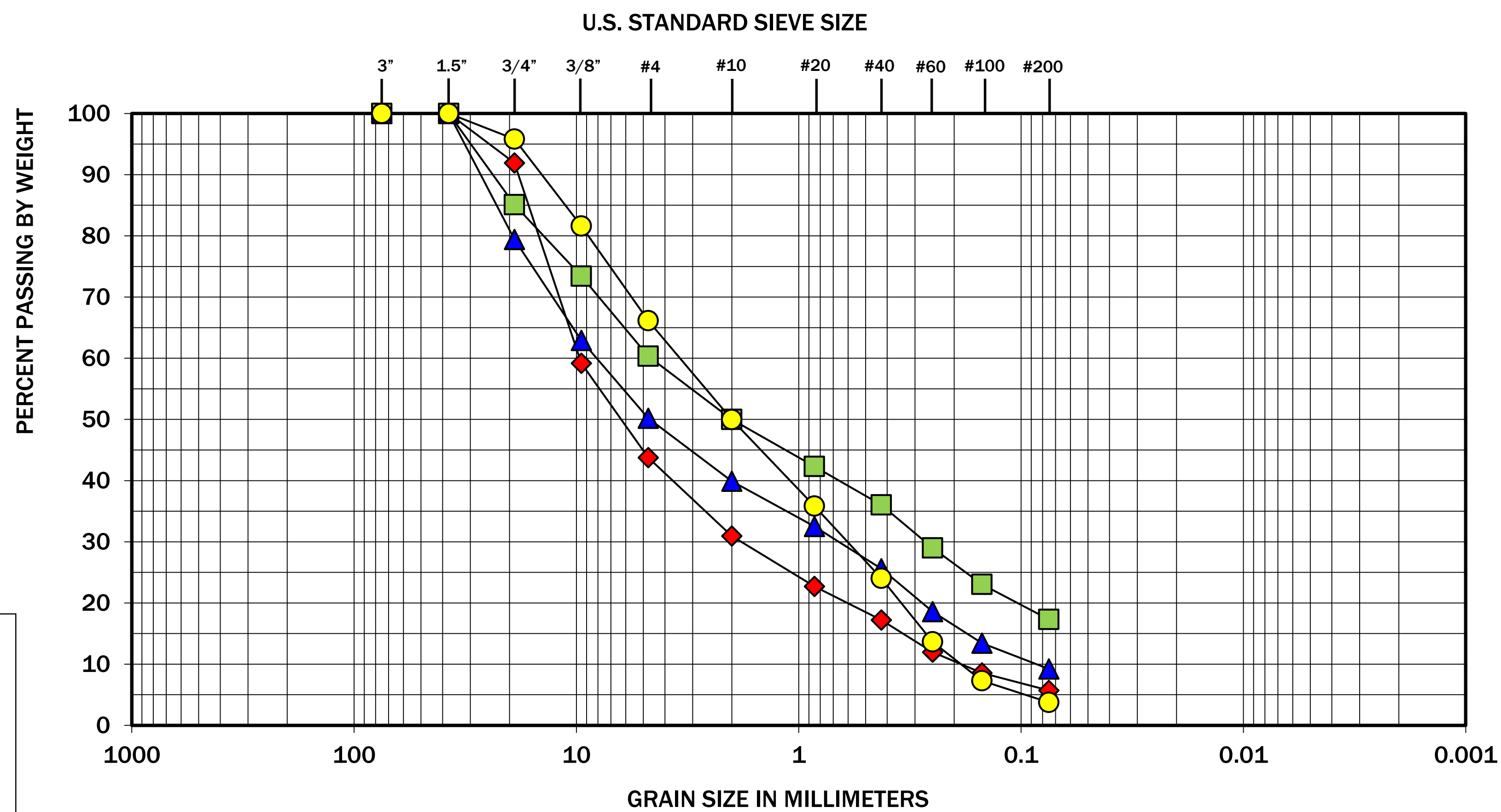
Sieve Analyses (SA)

Sieve analyses were performed on selected samples in general accordance with ASTM D 6913 and C 136. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted and classified in general accordance with the Unified Soil Classification System (USCS) and are presented in Figures B-1 through B-3.

Atterberg Limits Testing (AL)

Atterberg limits testing was performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and the plastic limit were estimated through a procedure performed in general accordance with ASTM D 4318. The results of the Atterberg limits testing are shown on the exploration logs in Appendix A at the respective sample depth and presented in Figures B-4 and B-5.

23237-002-00 Date Exported: 03/02/18



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	GEI-2	27.5	8	Fine gravel with silt and sand (GW-GM)
■	GEI-3	30	9	Silty fine to coarse sand with gravel (SM)
▲	GEI-7	15	9	Fine to coarse gravel with silt and sand (GP-GM)

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The grain size analysis results were obtained in general accordance with ASTM D 6913.

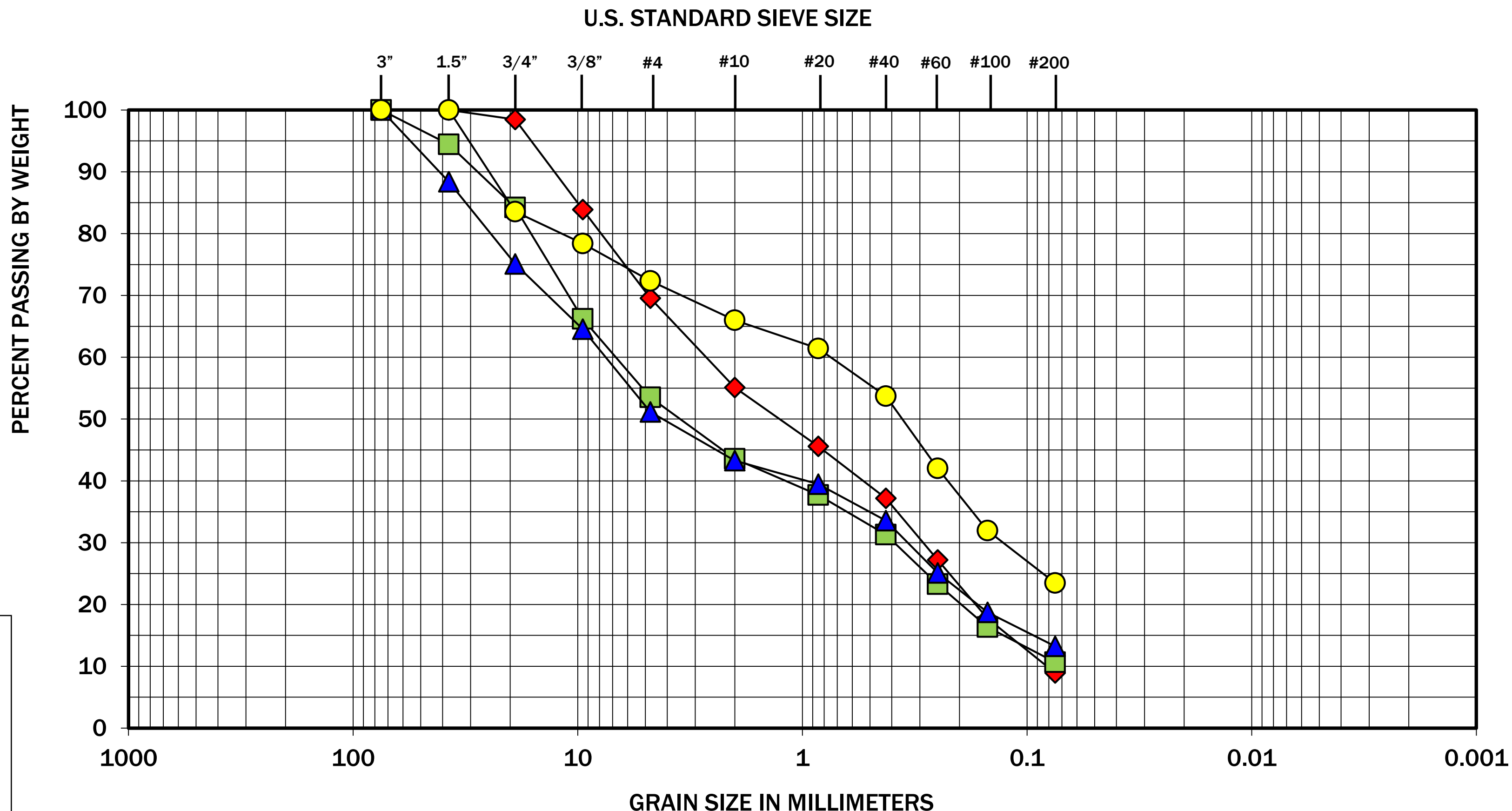
GeoEngineers

Building X
Redmond, Washington

Sieve Analysis Results

Figure B-1

23237-002-01 Date Exported: 12/13/18



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	GEI-19	12.5	10	Fine to coarse sand with silt and gravel (SP-SM)
■	GEI-19	15	8	Fine to coarse gravel with silt and sand (GP-GM)
▲	GEI-19	17.5	8	Silty fine to coarse gravel with sand (GM)
●	GEI-19	20	10	Silty fine to medium sand with gravel (SM)

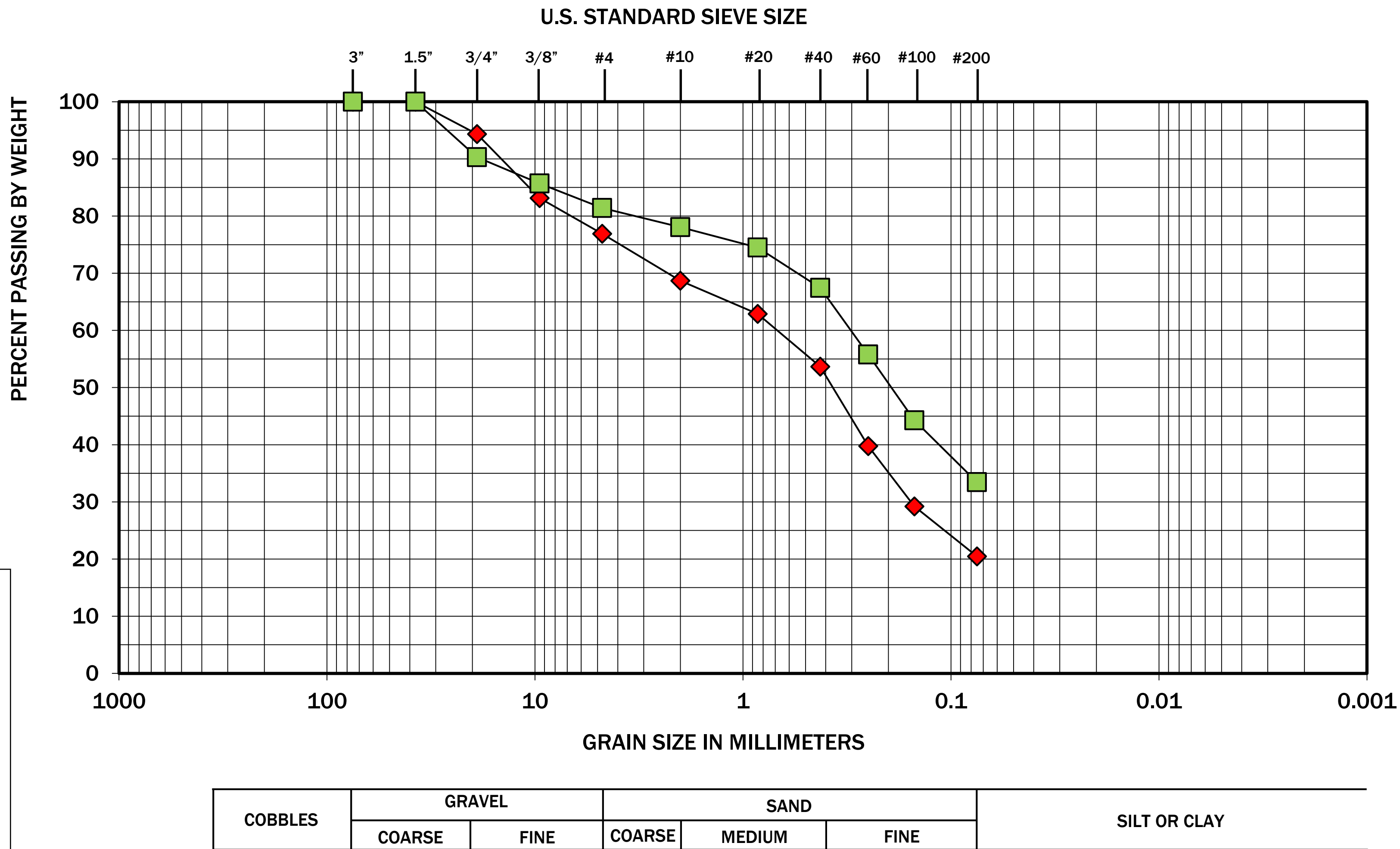
The grain size analysis results were obtained in general accordance with ASTM D 6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

GeoEngineers

Building X
Redmond, Washington

Sieve Analysis Results

Figure B-2



Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	GEI-19	20	9	Silty fine to medium sand with gravel (SM)
■	GEI-19	27.5	9	Silty fine to medium sand with gravel (SM)

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The grain size analysis results were obtained in general accordance with ASTM D 6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

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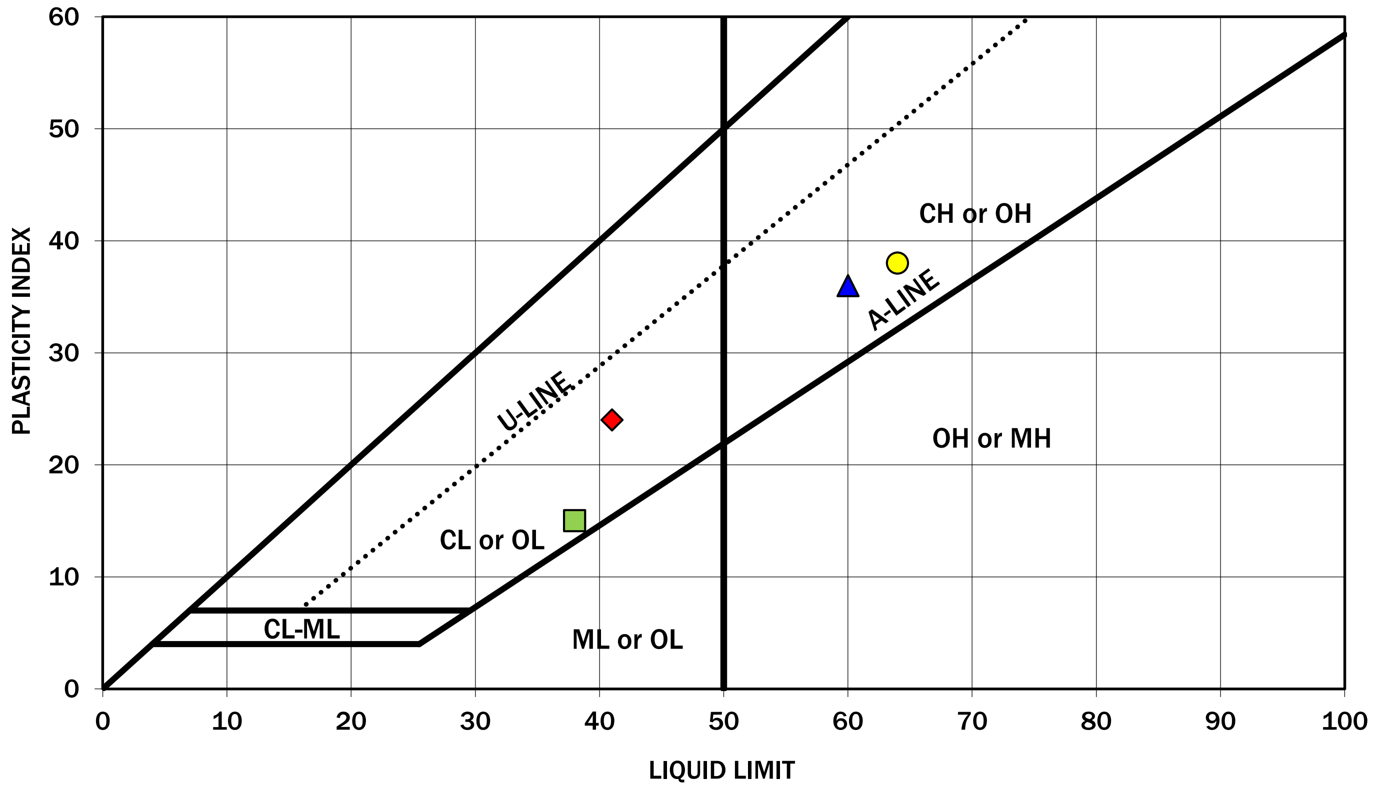
Building X
Redmond, Washington

Sieve Analysis Results

Figure B-3

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	GEI-2	80	21	41	24	Lean clay with sand (CL)
■	GEI-3	80	31	38	15	Lean clay with sand (CL)
▲	GEI-4	80	25	60	36	Fat clay (CH)
●	GEI-4	90	27	64	38	Fat clay (CH)

The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

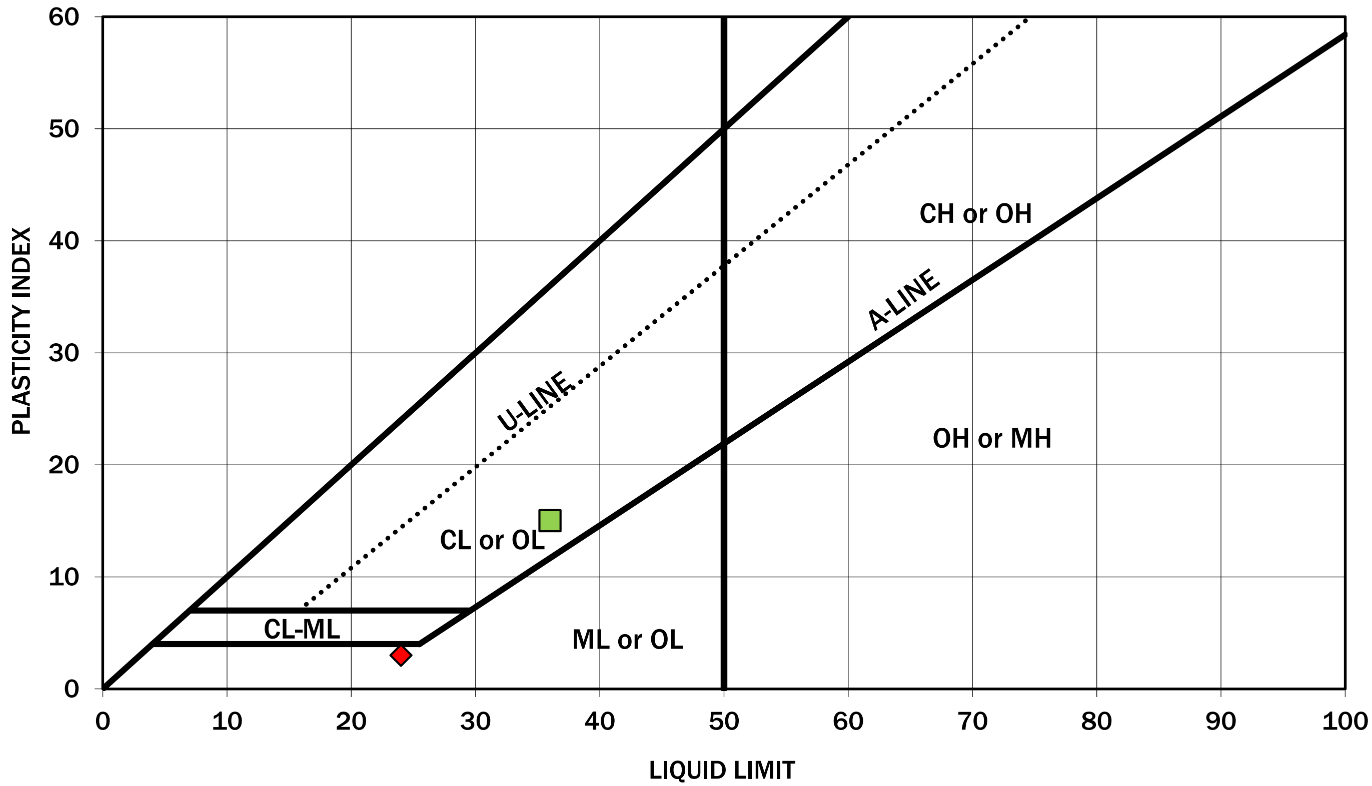
Building X
Redmond, Washington

GEOENGINEERS

Figure B-4

23237-002-00 Date Exported: 03/05/18

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	GEI-6	80	22	24	3	Silt (ML)
■	GEI-8	100	30	36	15	Lean clay with occasional sand (CL)

The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

Building X
Redmond, Washington

GeoEngineers

Figure B-5

23237-002-00 Date Exported: 03/05/18

APPENDIX C
Exploration Logs from Previous Studies

APPENDIX C
EXPLORATION LOGS FROM PREVIOUS STUDIES

Included in this section are logs from previous studies completed in the immediate vicinity of the project site.

- The logs of one boring (AB-05) and two borings with monitoring wells (AMW-01 and AMW-04) completed by Aspect Consulting in 2017 for the Mammoth project.

Aspect Consulting	Mammoth - 170655			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (SPN NAD83 ft)*	*Exploration Number*
	10201-10301 Willows Road, Redmond, Front lot, Bldg 2			E:1315677 N:255509 (est)	**AMW-01**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface (GS) Elev. (NAVD88)*	Ecology Well Tag No. BKA080
Cascade Drilling	Rotary drill rig	Autohammer; 300 lb hammer; 30" drop		62'(est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Curtis Askew	8.5 OD X 4.25 ID Hollow-Stem Auger	12/12/2017 to 12/13/2017		NA	29' (Static) 30' (ATD)

Depth (feet)	Elev. (feet)	Sample Type/ID	Blows/6"	Tests	Description
0	62				Asphalt
1	61				**ENGINEERED FILL** Loose, moist, orange-brown, slightly silty SAND (SP-SM); trace gravel
2	60				
3	59	S1	4, 4, 3	AMW-01-2.5 PID= 0.0 ppm RCRA 8 Geiger Counter= 50 CMP NWTPH-Dx Sheen= No 8260	
4	58				
5	57	S2	1, 2, 3	AMW-01-5.0 PID= 0.0 ppm RCRA 8 Geiger Counter= 25 CMP NWTPH-Dx Sheen= No 8260	Sand is orange brown
6	56				
7	55				
8	54	S3	3, 3, 8	AMW-01-7.5 PID= 0.0 ppm Geiger Counter= 50 CPM Sheen= No	
9	53				
10	52	S4	13, 14, 16	AMW-01-10.0 PID= 0.0 ppm Geiger Counter= 75 CPM Sheen= No	**CONSOLIDATED SAND AND GRAVEL** Medium dense, moist, light brown, gravelly, silty SAND (SM); fine to coarse sand, fine gravel
11	51				
12	50				
13	49				
14	48				
15	47	S5	16, 12, 13		
16	46				
17	45				
18	44				
19	43				
20	42	S6	12, 16, 15		Dense, moist to wet, gray, gravelly, silty SAND (SM); fine to coarse sand, fine gravel
21	41				
22	40				
23	39				
24	38				

Blows/foot ▲ Water Content (%) ● 0 10 20 30 40 50

Plastic Limit ├──┤ Liquid Limit

Legend

Sample Method: ◯ No Soil Sample Recovery; ▮ Split Barrel 3.25" X 2.375" (D&M)

Water Level: Static Water Level; Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: BG
Approved by:

Exploration Log AMW-01

Sheet 1 of 3

Aspect CONSULTING

Mammoth - 170655	Geotechnical Exploration Log	
Project Address & Site Specific Location	*Coordinates (SPN NAD83 ft)*	*Exploration Number*
10201-10301 Willows Road, Redmond, Front lot, Bldg 2	E:1315677 N:255509 (est)	**AMW-01**

Contractor	*Equipment*	*Sampling Method*	*Ground Surface (GS) Elev. (NAVD88)*	
Cascade Drilling	Rotary drill rig	Autohammer; 300 lb hammer; 30" drop	62'(est)	Ecology Well Tag No. BKA080
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Curtis Askew	8.5 OD X 4.25 ID Hollow-Stem Auger	12/12/2017 to 12/13/2017	NA	29' (Static) 30' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/foot ▲ Water Content (%)● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
			S7		13 14 15			Dense, moist to wet, gray, gravelly, silty SAND (SM); fine to coarse sand, fine gravel (continued)	
26	36								26
27	35								27
28	34								28
29	33	▼ 12/13/2017							29
30	32	▽ 12/13/2017	S8		11 12 15				30
31	31								31
32	30								32
33	29								33
34	28								34
35	27		S9		9 9 6			Loose to medium dense, wet, gray-brown SAND (SP); fine to medium sand, trace silt, trace gravel. Blow counts understated due to heave 35 to 45 feet	35
36	26								36
37	25								37
38	24								38
39	23								39
40	22		S10		2 2 4			Blow counts understated due to heave	40
41	21								41
42	20								42
43	19								43
44	18								44
45	17		S11		6 8 10			Blow counts understated due to heave	45
46	16								46
47	15								47
48	14								48
49	13								49

Legend

Plastic Limit ├──┤ Liquid Limit

Sample Method: ○ No Soil Sample Recovery; ▮ Split Barrel 3.25" X 2.375" (D&M)

Water Level: ▼ Static Water Level; ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: BG
Approved by:

Exploration Log AMW-01

Sheet 2 of 3

Aspect CONSULTING	Mammoth - 170655		Geotechnical Exploration Log	
	Project Address & Site Specific Location		*Coordinates (SPN NAD83 ft)*	*Exploration Number*
	10201-10301 Willows Road, Redmond, Front lot, Bldg 2		E:1315677 N:255509 (est)	**AMW-01**

Contractor	*Equipment*	*Sampling Method*	*Ground Surface (GS) Elev. (NAVD88)*	Ecology Well Tag No.
Cascade Drilling	Rotary drill rig	Autohammer; 300 lb hammer; 30" drop	62'(est)	BKA080
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Curtis Askew	8.5 OD X 4.25 ID Hollow-Stem Auger	12/12/2017 to 12/13/2017	NA	29' (Static) 30' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/foot ▲ Water Content (%)● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
			S12		15 18 21			**CONSOLIDATED TILL-LIKE DEPOSITS** Hard, wet, dark gray, sandy SILT (ML); trace gravel, diamict fabric	
51	11								51
52	10								52
53	9								53
54	8								54
55	7		S13		35 50/6"			**CONSOLIDATED SAND AND GRAVEL** Very dense, wet, gray, gravelly silty SAND (SM); medium to coarse sand, fine gravel	55
56	6								56
57	5								57
58	4								58
59	3								59
60	2		S14		20 50/4"			**CONSOLIDATED TILL-LIKE DEPOSITS** Hard, wet, dark gray, sandy SILT (ML); trace gravel, diamict fabric	60
61	1							Bottom of exploration at 60.8 ft. bgs.	61
62	0								62
63	-1								63
64	-2								64
65	-3								65
66	-4								66
67	-5								67
68	-6								68
69	-7								69
70	-8								70
71	-9								71
72	-10								72
73	-11								73
74	-12								74

Legend

Plastic Limit ├──┤ Liquid Limit

Sample Method
- No Soil Sample Recovery
- Split Barrel 3.25" X 2.375" (D&M)

Water Level
- Static Water Level
- Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: BG
Approved by:

Exploration Log AMW-01
Sheet 3 of 3

Mammoth - 170655

Geotechnical Exploration Log

Project Address & Site Specific Location	Coordinates (SPN NAD83 ft)	Exploration Number
10201-10301 Willows Road, Redmond, NW parking lot	E:1315685 N:254834 (est)	AMW-04

Contractor	Equipment	Sampling Method	Ground Surface (GS) Elev. (NAVD88)	Ecology Well Tag No.
Cascade Drilling	Rotary drill rig	Autohammer; 300 lb hammer; 30" drop	91'(est)	BKF079

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Curtis Askew	8.5 OD X 4.25 ID Hollow-Stem Auger	12/11/2017	NA	5.8' (Static) 22' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/6"	Tests	Description
0	91					Asphalt
1	90					**CONSOLIDATED SAND AND GRAVEL** Medium dense, moist, gray, slightly gravelly, silty SAND (SM); fine to medium sand
2.5–4	88–87		S1	5, 5, 7	AMW-04-2.5 PID= 0.1 ppm RCRA 8 Geiger Counter= 50 CPM NWTPH-Dx Sheen= No 8260	
5–6.5	86–85	12/15/2017 (Static Water Level)	S2	3, 5, 7	AMW-04-5.0 PID= 0.1 ppm Geiger Counter= 50 CPM Sheen= Very Slight	
7.5–9	83–82		S3	5, 11, 12	AMW-04-7.5 PID= 0.1 ppm Geiger Counter= 50 CPM Sheen= No	Becomes light brown
10–11.5	81–80		S4	15, 16, 30	AMW-04-10.0 PID= 0.1 ppm Geiger Counter= 75 CPM Sheen= No	Becomes dense. With cobbles.
12.5–14	78–77		S5	8, 16, 20	RCRA 8 AMW-04-12.5 PID= 0.1 ppm NWTPH-Dx Geiger Counter= 100 CPM 8260 Sheen= No	Rusty staining around gravels 12.5 to 16 ft bgs
15–16.5	76–75		S6	15, 27, 23		
20–21	71–70		S7	25, 50/6"		**CONSOLIDATED TILL-LIKE DEPOSITS** Hard, moist, dark blue gray, sandy SILT (ML); fine sand, diamict fabric
22	69	12/11/2017 (Water Level ATD)				
23	68					
24	67					

Legend

Sample Method:
- No Soil Sample Recovery
- Split Barrel 3.25" X 2.375" (D&M)

Plastic Limit ⊢—⊣ Liquid Limit

Water Level:
- Static Water Level
- Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: BG
Approved by:

Exploration Log AMW-04

Sheet 1 of 2

Aspect CONSULTING

Mammoth - 170655

Geotechnical Exploration Log

Project Address & Site Specific Location	Coordinates (SPN NAD83 ft)	Exploration Number
10201-10301 Willows Road, Redmond, NW parking lot	E:1315685 N:254834 (est)	**AMW-04**

Contractor	Equipment	Sampling Method	Ground Surface (GS) Elev. (NAVD88)	Ecology Well Tag No.
Cascade Drilling	Rotary drill rig	Autohammer; 300 lb hammer; 30" drop	91'(est)	BKF079

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Curtis Askew	8.5 OD X 4.25 ID Hollow-Stem Auger	12/11/2017	NA	5.8' (Static) 22' (ATD)

Depth (feet)	Elev. (feet)	Sample Type/ID	Blows/6"	Description
25–26.5	66–64.5	S8	10 20 20	**CONSOLIDATED TILL-LIKE DEPOSITS** Hard, moist, dark blue gray, sandy SILT (ML); fine sand, diamict fabric (continued)
30	61	S8	16 23 46	1-inch beds silty sand, faint stratification in silt
35	56	S10	30 50/6"	Gravelly
40	51	S11	14 50/6"	**CONSOLIDATED SAND AND GRAVEL** Very dense, wet, dark gray, SAND (SP); medium to coarse sand. HEAVE
41	50			Bottom of exploration at 41 ft. bgs.
				Note: Well installed in borehole 7 feet north of AMW-04

Depth (feet) / Elev. (feet): 26 / 65, 27 / 64, 28 / 63, 29 / 62, 30 / 61, 31 / 60, 32 / 59, 33 / 58, 34 / 57, 35 / 56, 36 / 55, 37 / 54, 38 / 53, 39 / 52, 40 / 51, 41 / 50, 42 / 49, 43 / 48, 44 / 47, 45 / 46, 46 / 45, 47 / 44, 48 / 43, 49 / 42

Columns: Exploration Completion and Notes; Blows/foot ▲ Water Content (%) ● 0 10 20 30 40 50; Tests; Material Type

Legend

Sample Method: No Soil Sample Recovery; Split Barrel 3.25" X 2.375" (D&M)

Plastic Limit ⊢—⊣ Liquid Limit

Water Level: Static Water Level; Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: BG
Approved by:

Exploration Log AMW-04

Aspect CONSULTING	Mammoth - 170655			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (1315633.871)*	*Exploration Number*
	10201-10301 Willows Road, Redmond, Loading dock, Bldg 2			E:255325.4 N:255328 (est)	**AB-05**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface (GS) Elev. (NAVD88)*	
Cascade Drilling	Rotary drill rig	Autohammer; 300 lb hammer; 30" drop		59'(est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Curtis Askew	8.5 OD X 4.25 ID Hollow-Stem Auger	12/12/2017		NA	24.5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/6"	Tests	Description	Depth (ft)
0	59					Asphalt	
1	58					**ENGINEERED FILL** Medium dense, moist, brown, gravelly, silty SAND (SM)	1
2	57						2
3	56		S1	5 5 6	AB-05-2.5 PID= 0.3 ppm RCRA 8 Geiger Counter= 50 CPM NWTPH-Dx Sheen= No 8260		3
4	55						4
5	54		S2	5 6 11	AB-05-5.0 PID= 0.0 ppm Geiger Counter= 50 ppm Sheen= No		5
6	53						6
7	52						7
8	51		S3	11 14 9	AB-05-7.5 PID= 0.0 ppm Geiger Counter= 50 CPM Sheen= No		8
9	50						9
10	49		S4	7 6 8	AB-05-10.0 PID= 0.0 ppm RCRA 8 Geiger Counter= 50 CPM NWTPH-Dx Sheen= Slight 8260	**CONSOLIDATED SAND AND GRAVEL** Medium dense, moist, light gray-brown, gravelly, silty SAND (SM); fine to coarse sand	10
11	48						11
12	47						12
13	46						13
14	45						14
15	44		S5	19 15 15		Medium dense, moist, light gray-brown, gravelly, slightly silty SAND (SP-SM); fine to coarse sand	15
16	43						16
17	42						17
18	41						18
19	40						19
20	39		S6	8 13 18		Dense, moist, light gray-brown, gravelly, silty SAND (SM); fine to coarse sand	20
21	38						21
22	37						22
23	36						23
24	35	∇ 12/12/2017					24

Blows/foot ▲ Water Content (%) ● 0 10 20 30 40 50

Plastic Limit ⊢⊣ Liquid Limit

Legend

Sample Method
- ○ No Soil Sample Recovery
- ▮ Split Barrel 3.25" X 2.375" (D&M)

Water Level
- ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: BG
Approved by:

Exploration Log AB-05

Sheet 1 of 2

Aspect CONSULTING	Mammoth - 170655			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (1315633.871)*	*Exploration Number*
	10201-10301 Willows Road, Redmond, Loading dock, Bldg 2			E:255325.4 N:255328 (est)	**AB-05**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface (GS) Elev. (NAVD88)*	
Cascade Drilling	Rotary drill rig	Autohammer; 300 lb hammer; 30" drop		59'(est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Curtis Askew	8.5 OD X 4.25 ID Hollow-Stem Auger	12/12/2017		NA	24.5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/foot ▲ Water Content (%)● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
			S7		13 18 30			Dense, moist, light gray-brown, gravelly, silty SAND (SM); fine to coarse sand (continued) Light brown 25-26 ft bgs	
26	33								26
27	32								27
28	31								28
29	30					Water sample AB-05-20171212			29
30	29		S8		28 28 25			**CONSOLIDATED TILL-TILL DEPOSITS** Very dense, moist, dark gray, sandy, gravelly SILT (ML); medium sand, fine and coarse gravel, diamict fabric	30
31	28								31
32	27								32
33	26								33
34	25								34
35	24		S9		22 50/4"				35
36	23								36
37	22								37
38	21								38
39	20								39
40	19		S10		32 50/4"				40
41	18							Bottom of exploration at 40.8 ft. bgs.	41
42	17								42
43	16								43
44	15								44
45	14								45
46	13								46
47	12								47
48	11								48
49	10								49

Legend

Sample Method
- No Soil Sample Recovery
- Split Barrel 3.25" X 2.375" (D&M)

Plastic Limit ├──┤ Liquid Limit

Water Level
- Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: BG
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Exploration Log AB-05

Sheet 2 of 2

APPENDIX D
Geophysical Testing

APPENDIX D
GEOPHYSICAL TESTING

Shear wave velocity testing and vibration monitoring was completed at the project site using microtremor array measurements. The testing was completed by Oyo Pacific from October 2 through 5, 2018.

The locations of the testing and monitoring locations were estimated by taping/pacing from existing site features and are shown in Figures 1.2 and 1.8 of the geophysical testing report. The details of the geophysical testing are presented in the report compiled by Oyo Pacific.

BUILDING X, SHEAR WAVE VELOCITY AND VIBRATION MONITORING REPORT

OYO Corporation, Pacific
919 Pale San Vitores Rd., Tumon
Guam 96913, U.S.A.

Table of contents

1. Outline

OYO Corporation, Pacific conducted 2D high resolution surface wave method (multi-channel analysis of surface waves: MASW), microtremor array measurements (MAM) and vibration monitoring (VM) at 10301 Willows Rd, Redmond, WA. The purposes of the MASW and MAM are the estimation of S-wave velocity profiles down to a depth of about 30 and 400 feet below ground surface respectively. The purpose of the vibration monitoring is the understanding of vibration characteristics at the site. This report summarizes the methodology, instrumentation, data acquisition, analysis and results of the geophysical investigations and the vibration monitoring.

1.1 Investigation site

Figures 1.1 shows the investigation site.

1.2 Summary of measurements

Table 1.1 summarizes the quantity and the investigation depth of MASW and MAM conducted under the investigation.

Table 1.1 Amount of measurements

Measurements	Number of lines/arrays	Investigation depth (feet)
2D high resolution surface wave method (MASW)	6	30
Shallow microtremor array measurements (MAM)	3	200
Deep microtremor array measurements (MAM)	1	400
Vibration monitoring (VM)	3	-

1.3 Configuration of survey lines, arrays and vibration monitoring

Tables 1.2 to 1.4 summarize survey line lengths, array sizes and measurement specification of MASW, MAM and VM respectively. Figures 1.2 to 1.8 show the configurations of MASW lines, MAM arrays and VM lines respectively.

Table 1.2 Survey line lengths of 2D high resolution surface wave method (MASW).

Line name	Survey line length (feet)	Number of receivers	Number of shots	Receiver interval (feet)	Shot interval (feet)
A	156	40	21	4	8
B	156	40	21	4	8
C	156	40	21	4	8
D	156	40	21	4	8
E	156	40	21	4	8
F	156	40	21	4	8
Total	936	240	126	-	-

Table 1.3 Array sizes of shallow and deep microtremor array measurements (MAM).

Measurements	Array name	Investigation depth (feet)	Array size (feet)	Min. and Max. receiver separation (feet)	Number of receivers	Number of receiver separations
Shallow arrays	G	200	300	25.0 to 459.6	10	24
	H	200	300	25.0 to 424.3	10	24
	I	200	300	25.0 to 424.3	10	24
Deep array	J	400	1593	230.5 to 1592.6	10	44
Total		-	-	-	40	-

Table 1.4 Measurement specification of vibration monitoring.

Line name	Site	Number of accelerometers	Line length (feet)	Measurement purpose
K	Parking lot	4	149	Vibration from a speed bump
L	North entrance	4	300	Noisy and quiet time
M	South entrance	4	300	Noisy time
Total	-	12	749	-

1.4 Date of data acquisition

Data acquisition was performed on the dates and times shown in Table 1.5 below. MASW and shallow arrays of MAM were conducted during the daytime. A deep array of MAM was conducted at night. Vibration monitoring was conducted both during the daytime and at night.

Table 1.5 Date and time of data acquisition.

Measurements	Date of acquisition	Time of acquisition
2D high resolution surface wave method (MASW)	Oct. 2, 2018	9:00 am to 3:00 pm
Shallow microtremor array measurements (MAM)	Oct. 3, 2018	9:00 am to 3:00 pm
Deep microtremor array measurements (MAM)	Oct. 4, 2018	9:00 pm to 3:00 am (Oct. 5)
Vibration monitoring (VM)	Oct. 4, 2018	6:00 am to 3:00 am (Oct. 5)

1.5 Persons in charge and certification

Professional Engineer:

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kbull@oyopacific.com

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Certified P.E. Seal

Geophysicist(s):

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Geophysicist
khayashi@geometrics.com

OYO Corporation
2190 Fortune Drive
San Jose, CA, 95131
Tel : 408-954-0522
Fax : 408-954-0902

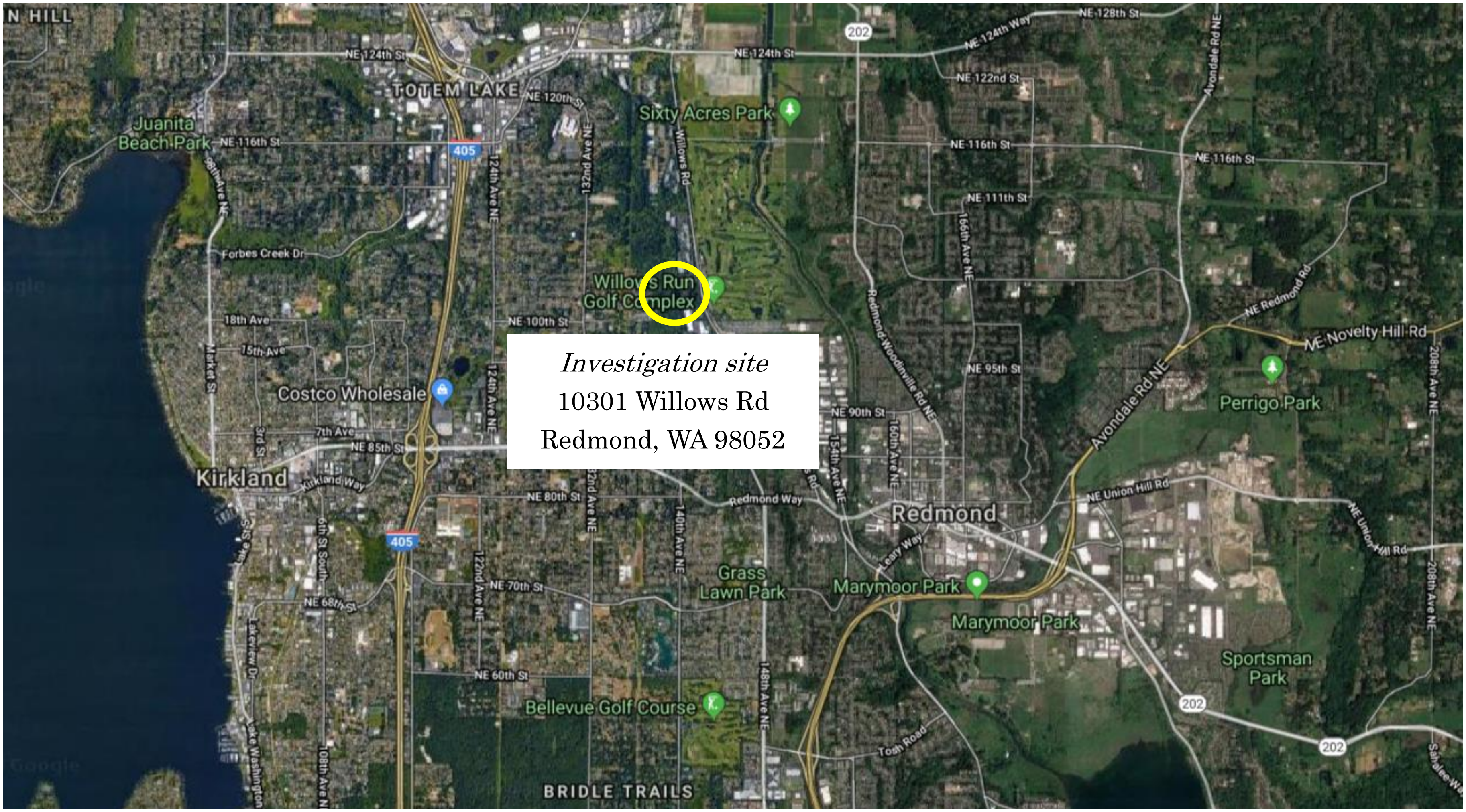


Figure 1.1 Site of investigation.

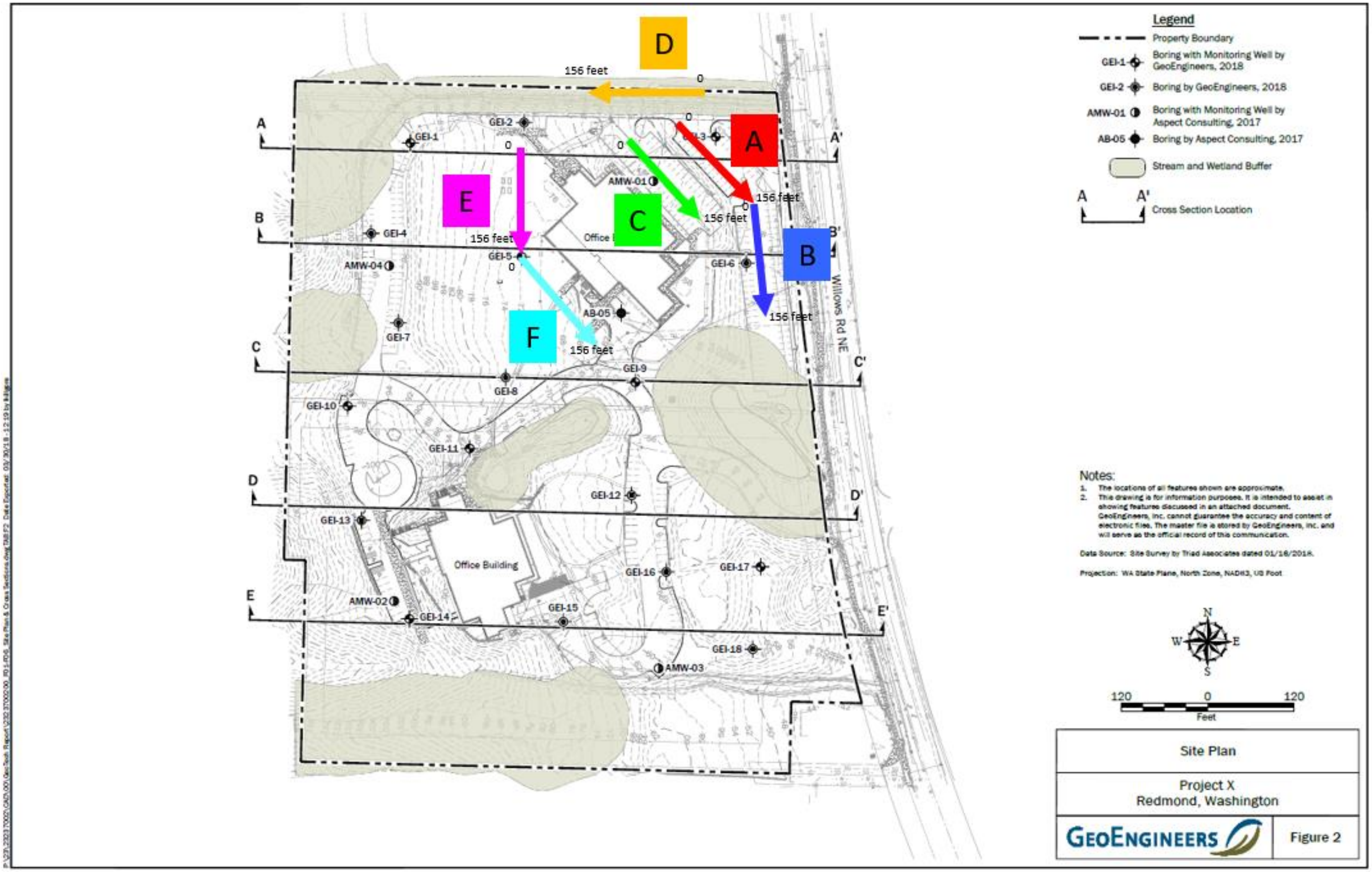


Figure 1.2 Configuration of 2D high resolution surface wave method (MASW).

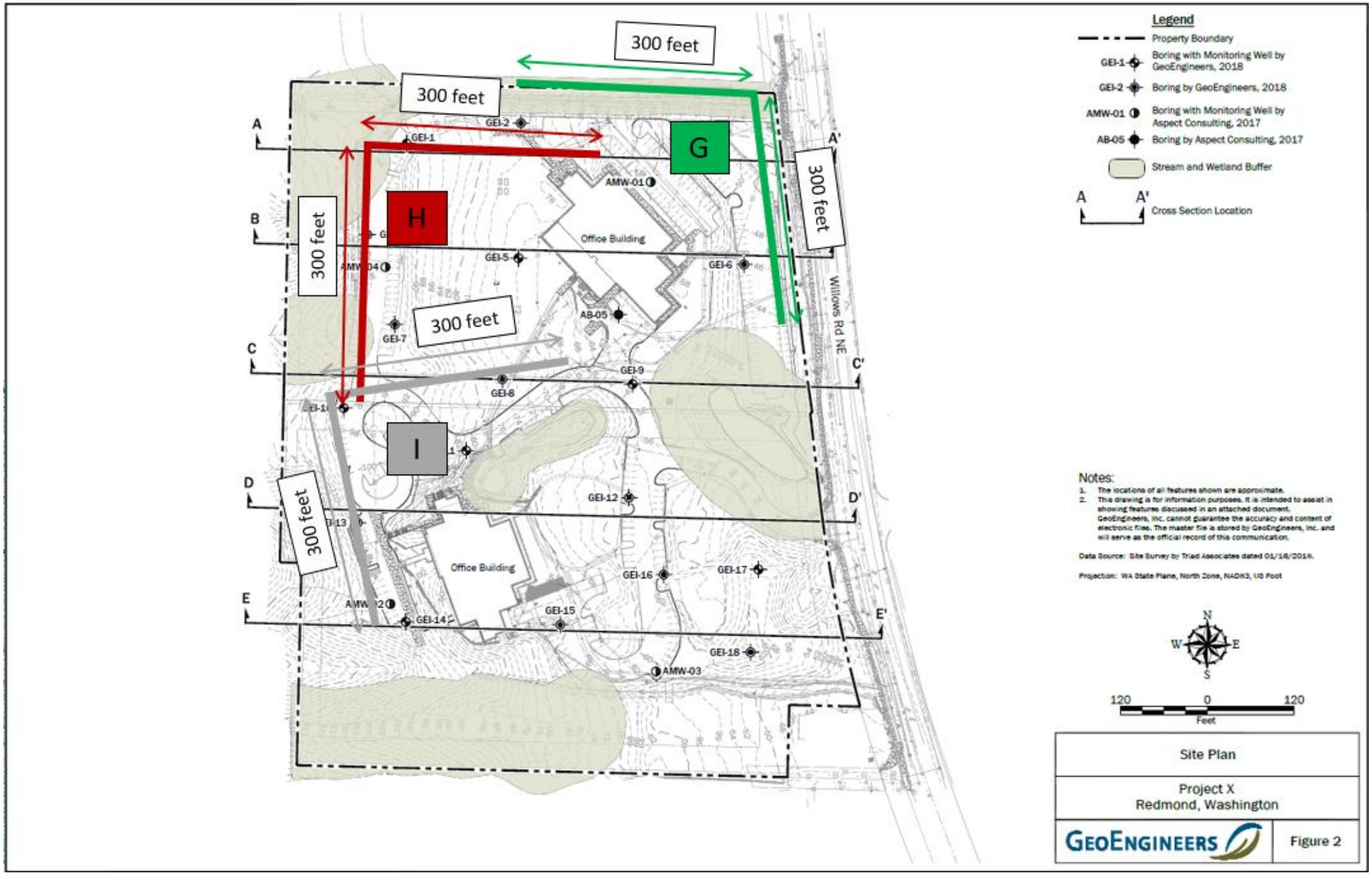


Figure 1.3 Configuration of shallow microtremor array measurements (MAM).

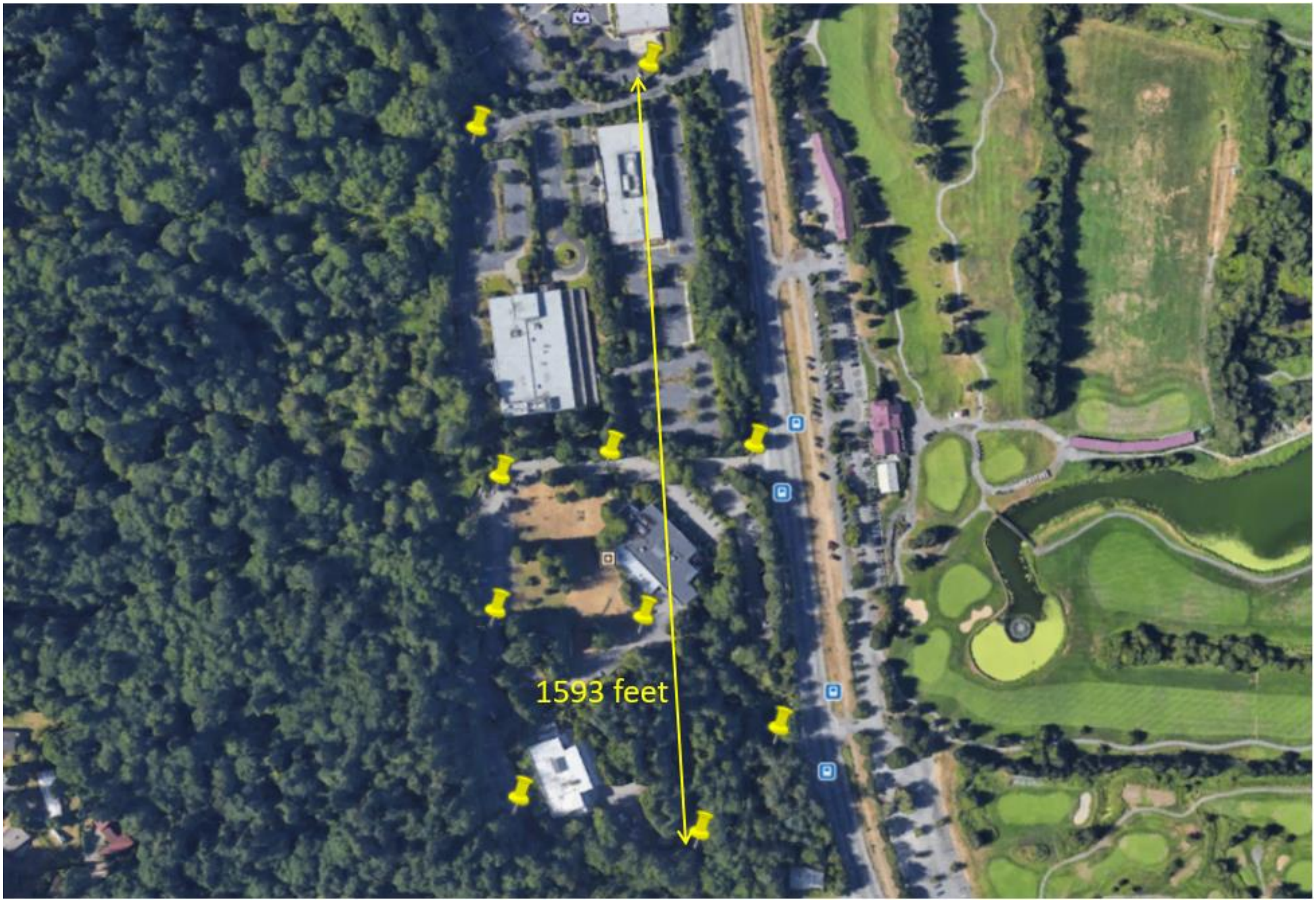


Figure 1.4 Configuration of large microtremor array measurements (MAM).

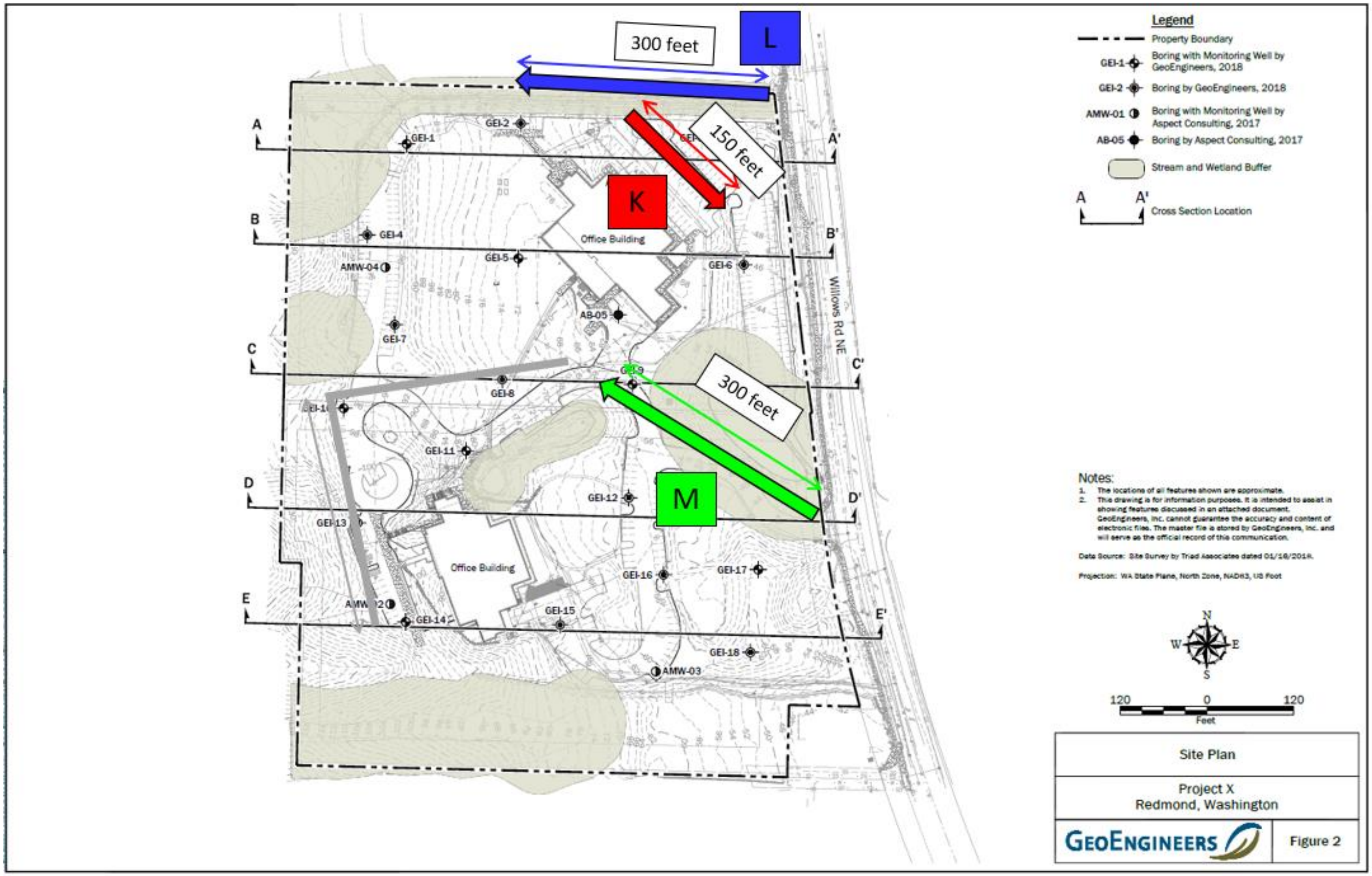


Figure 1.5 Configuration of vibration monitoring (VM).

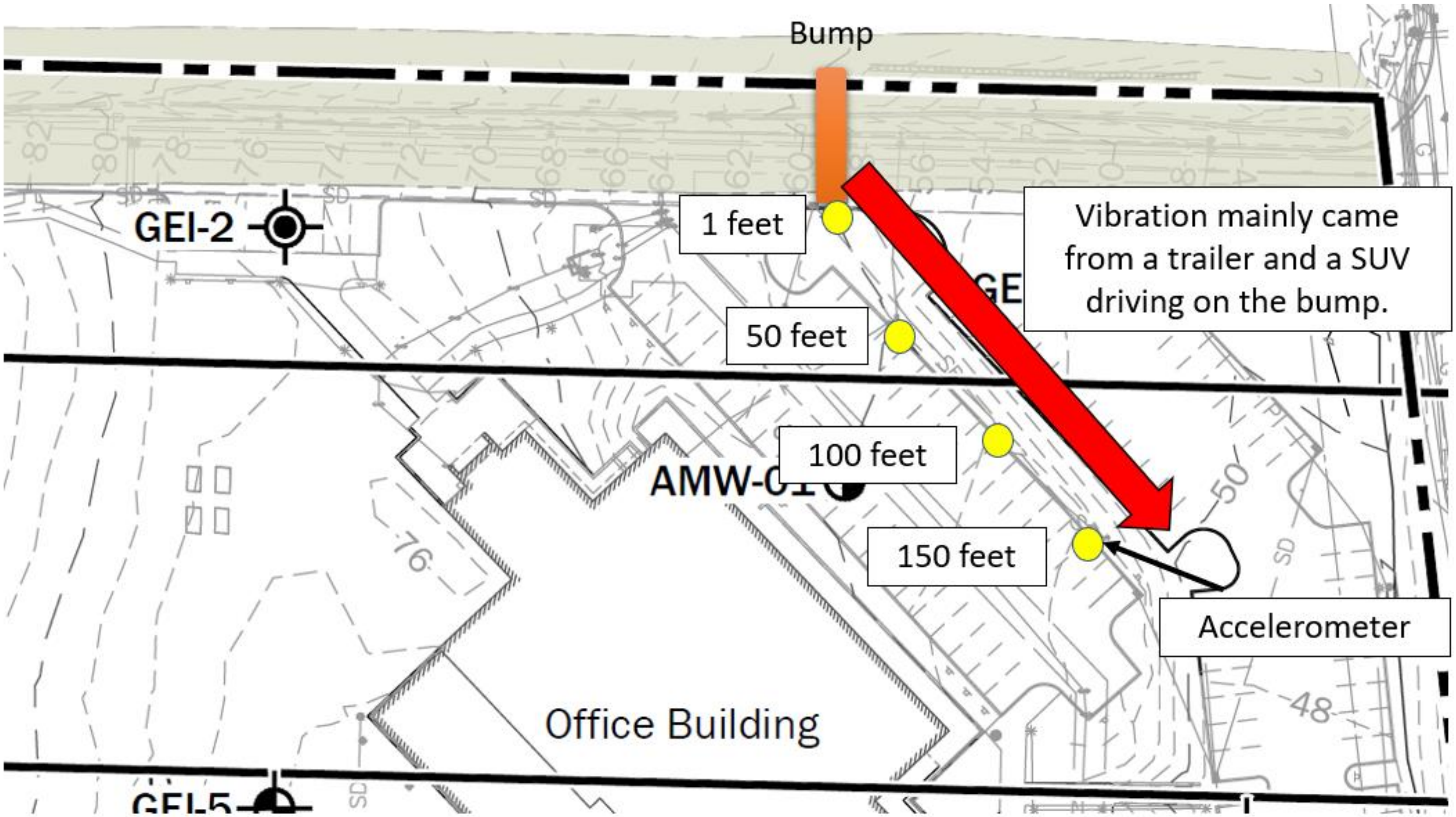


Figure 1.6 Configuration of vibration monitoring (VM) of K-line.

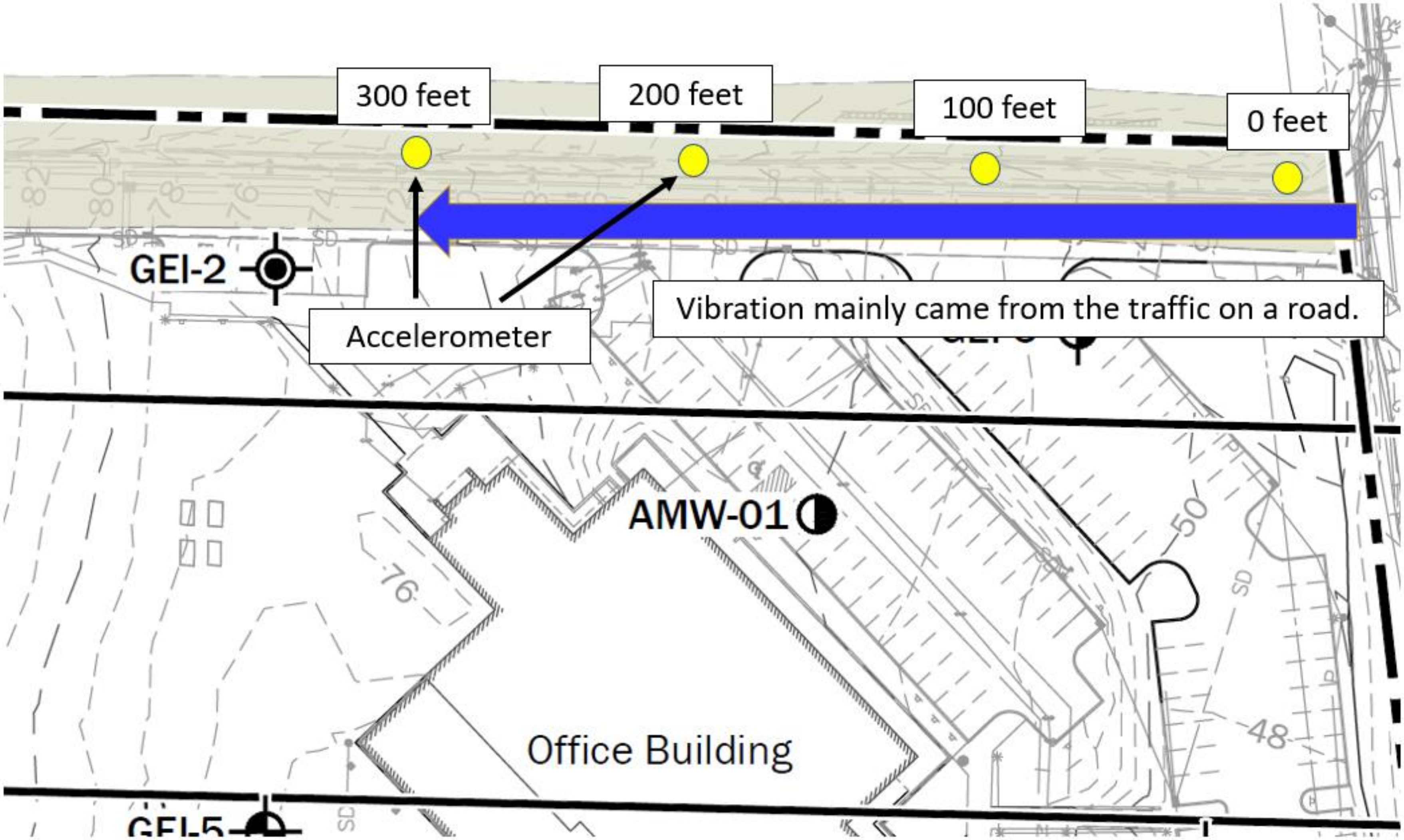


Figure 1.7 Configuration of vibration monitoring (VM) of L-line.

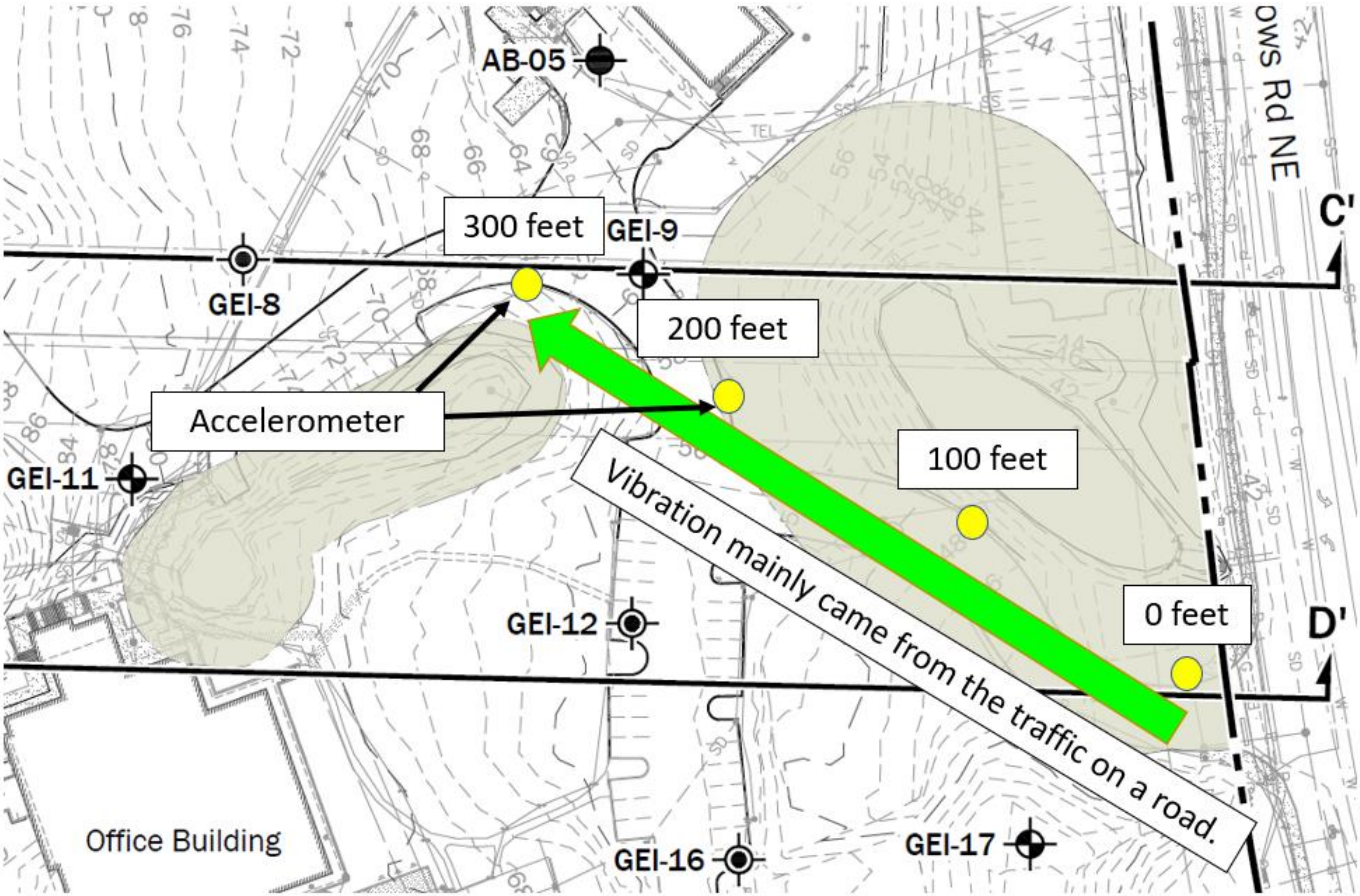


Figure 1.8 Configuration of vibration monitoring (VM) of M-line.

2. 2D high resolution surface wave method (MASW)

2.1 Methodology

A surface wave (Rayleigh wave) is an elastic wave propagating along the ground surface and its energy concentrates near the ground surface. The propagation velocity of the surface wave strongly depends on the S-wave velocity of the ground. If the subsurface S-wave velocity varies with depth, the propagating velocity also varies with its frequency or its wavelength. This characteristic is called dispersion. The sub-surface S-wave velocity structure can be estimated by the analysis of surface wave dispersion. The surface wave method is a geophysical exploration methodology in which the sub-surface S-wave velocity structure is estimated from the dispersion characteristics of the surface waves. Figure 2.1 shows a schematic diagram of the surface wave method.

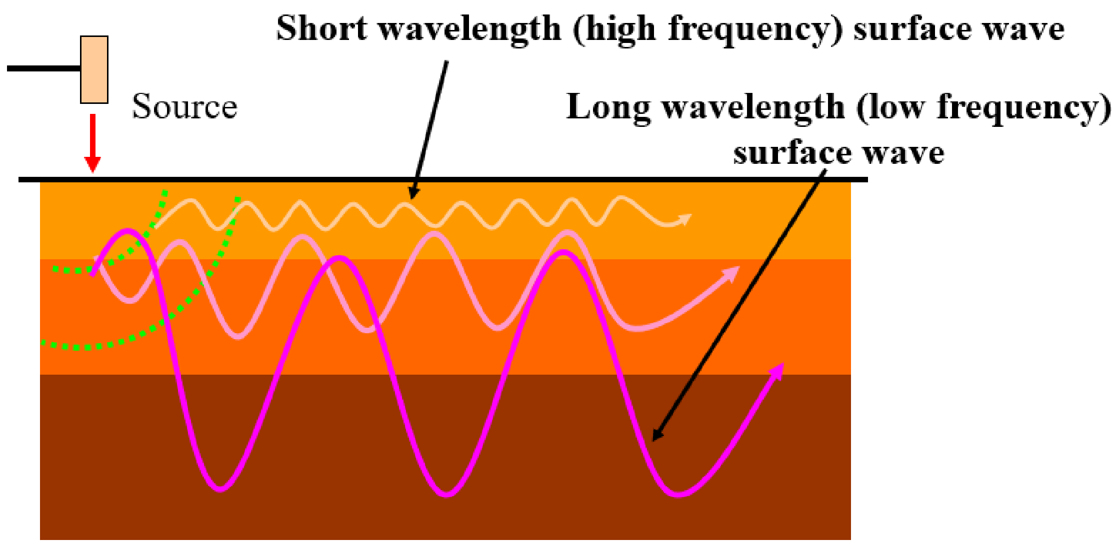


Figure 2.1 Schematic diagram of the surface wave method. Short wavelength (high frequency) surface waves only propagate at shallow depths and reflects shallow S-wave velocity. Whereas, long wavelength (low frequency) surface wave penetrates into deeper depths and reflects deep S-wave velocity.

Recently, the surface wave methods using active sources, such as a sledgehammer or a weight drop, have been applied to the delineation of shallow S-wave velocity structures. We will use the term “multi-channel analysis of surface waves (MASW)” to refer to the active surface wave methods.

2.2 Instrumentation

A seismograph (Geode) made by Geometrics Inc. and 24 geophones with a natural frequency of 4.5 Hz were used for data acquisition. Figure 2.2 shows the Geode and its main specifications are summarized in Table 2.1.

Figure 2.2 Seismographs (Geode) used in the data acquisition of MASW.

Table 2.1 Specification of Geode and sensors

Sensor	4.5 Hz Geophone (24 sensors were used)
Number of channels	24
Impedance	20 kΩ
Gain	24db、36db
Data length	Maximum 16384
Frequency Response	1.75Hz ~20KHz
Pre-trigger	Maximum 16384
A/D Converter	24 bits
Sampling time	0.02, 0.3125, 0.0625, 0.125, 0.25, 0.5, 1.0, 2.0, 4.0, 8.0, 16.0 msec
Weight	Approx. 3.6 kg

2.3 Data acquisition

The multichannel analysis of surface waves (MASW) was used to determine a 2D S-wave velocity depth profile for each of the 6 survey lines described above. MASW is an active surface wave method utilizing an impulsive source (Xia et al., 1999). MASW survey was conducted at each survey line using a 24-channel seismographic system with a linear array of vertical-component 4.5 Hz geophones, 4 feet geophone spacing, 92 feet spread length. Acquisition parameters were chosen to provide adequate spatial resolution and depth penetration. Theoretical minimum wavelength and depth resolution are determined by geophone spacing, while maximum wavelength and depth penetration are determined by geophone resonant frequency and spread length. A relatively small geophone spacing (4 feet) was used in order to provide sufficient vertical resolution to resolve details of the near surface velocity structure. When using the active surface wave method, a spread length of about 2 times the target depth is required. The target depth of this survey was about 30 feet, and a spread length of 92 feet was used. Figure 2.3 shows the schematic diagram of data acquisition of MASW and Figure 2.4 shows the example of source-receiver geometry.

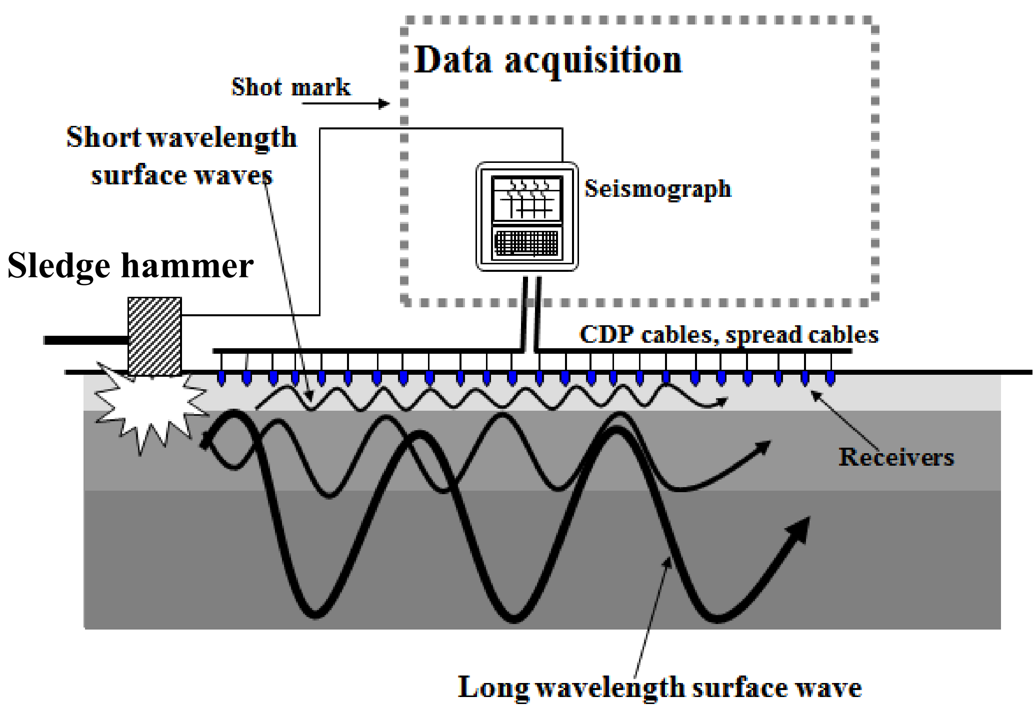


Figure 2.3 Schematic diagram of data acquisition of MASW.

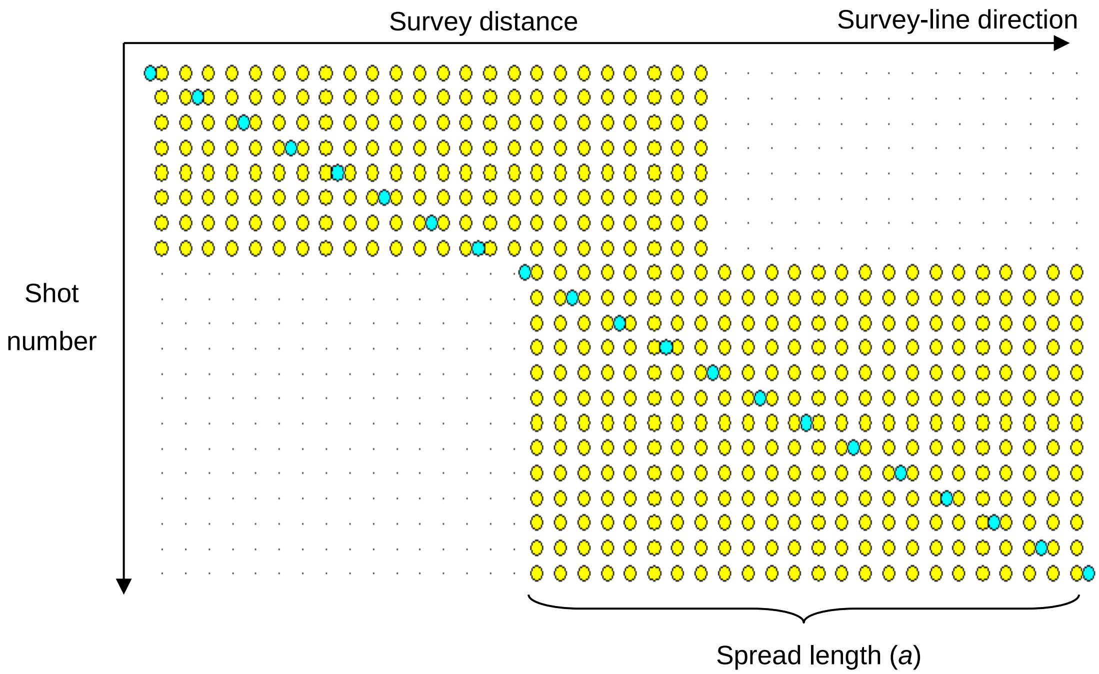


Figure 2.4 Example of source-receiver geometry. Yellow circles and blue circles represent the location of receivers and sources respectively.

2.4 Analysis

Recorded shot gathers were analyzed in terms of a multi-channel analysis of surface-waves (MASW). Data processing of MASW is summarized as follows and their flows are shown in Figure 2.5. During the processing, a CMP cross-correlation analysis (Hayashi and Suzuki, 2004) method was applied to shot gathers and CMP cross-correlation (CMPCC) gathers are calculated in order to improve lateral resolution of S-wave velocity profiles. The procedure of MASW including CMPCC analysis is summarized as follows.

1) Calculation of CMPCC

a) Figure 2.6 shows an example of raw shot gather. In each shot gather, cross-correlations are calculated for every pairs of two traces. For example, 276 cross-correlations ($={}_{24}C_2$) are calculated from a shot gather that includes 24 traces.

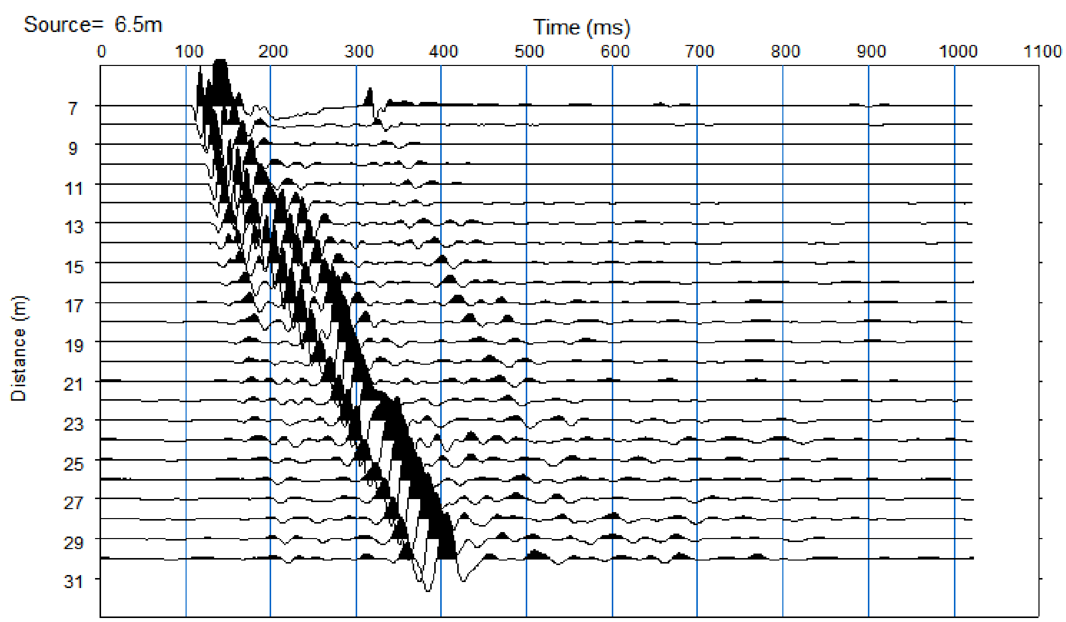


Figure 2.6 Example of raw shot gather.

b) From cross-correlations for each pair of all shot gathers, correlations having a common mid-point are grouped together.

c) In each common mid-point, cross-correlations that have an equal spacing are stacked in the time domain. Even if each source wavelet and its phases are different, cross-correlations can be stacked because the correlation stores only phase-differences between two traces. The phase-differences contained in the source wavelet has been removed by cross-correlation if we assume two traces only contain waves propagating in the horizontal direction.

d) The cross-correlations that have different spacing should not be stacked in the time domain. The different-spacing cross-correlations are ordered with respect to their spacing at each common mid-point. Resultant cross-correlation gathers resembles the shot gathers. However, it contains only characteristic phase differences in each CMP location, and can be handled as shot gathers in the phase-velocity analysis. We call this a CMPCC gather. Figure 2.7 shows an example of a CMPCC gather.

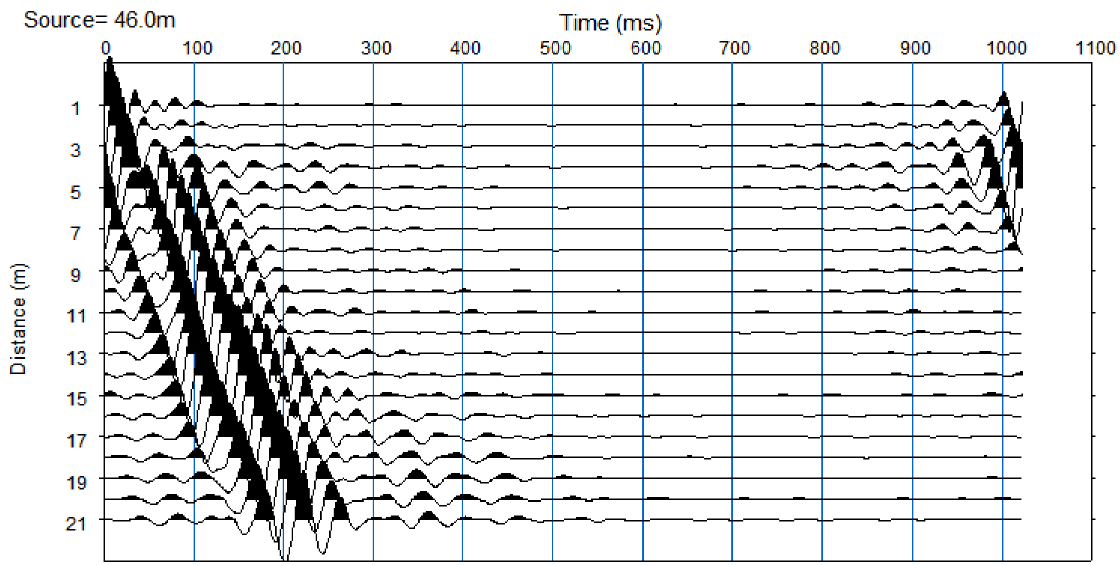


Figure 2.7 Example of a CMPCC gather.

2) Phase velocity calculation

a) The multi-channel analysis of surface waves (MASW) is applied to the CMPCC gathers for calculating phase-velocities. First, each trace is transformed into the frequency domain by Fast Fourier Transform (FFT). Then, frequency-domain data is integrated over the spacing with respect to phase-velocities. Using these procedures, the CMPCC gathers in the spacing and time domain can be transformed into phase-velocity and frequency domain directly. Figure 2.8 shows an example of phase-velocity image in frequency domain. In the figure,

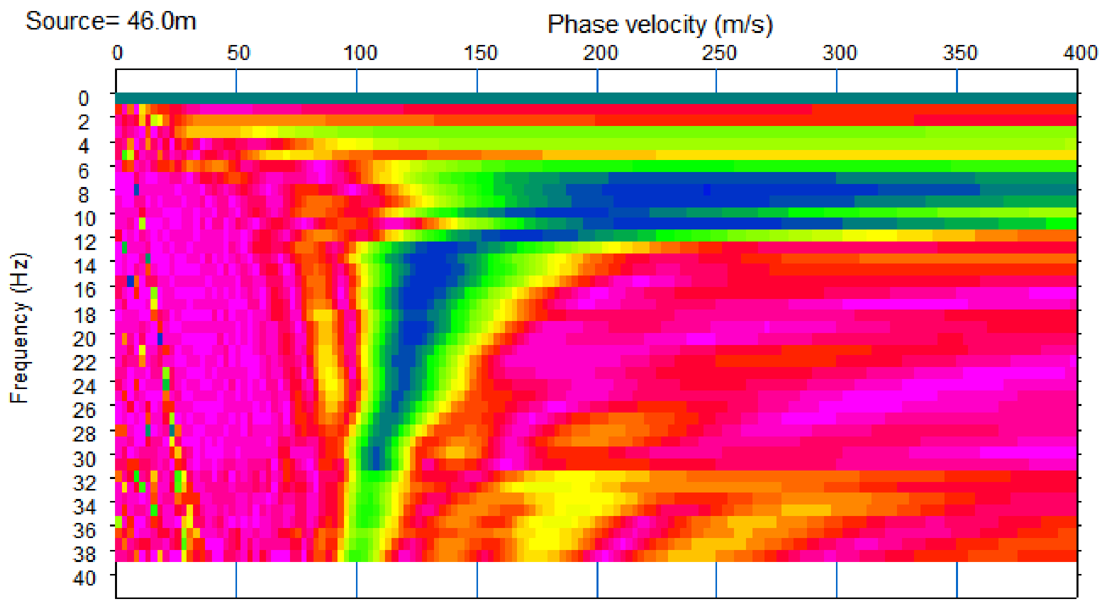


Figure 2.8 Example of phase-velocity image in frequency domain.

difference of color indicates difference in amplitude. Blue color indicates large amplitude and red color indicates small amplitude. Maximum amplitude phase velocity is picked at each frequency and a series of phase velocities result in a dispersion curve.

b) Phase velocities are determined as the maximum amplitude in each frequency. This series of phase velocities defines a dispersion curve.

3) Dispersion curve

Phase velocity images are calculated for all CMPCC gathers and a dispersion curve is determined in each phase velocity image. Figure 2.9 shows an example of dispersion curves.

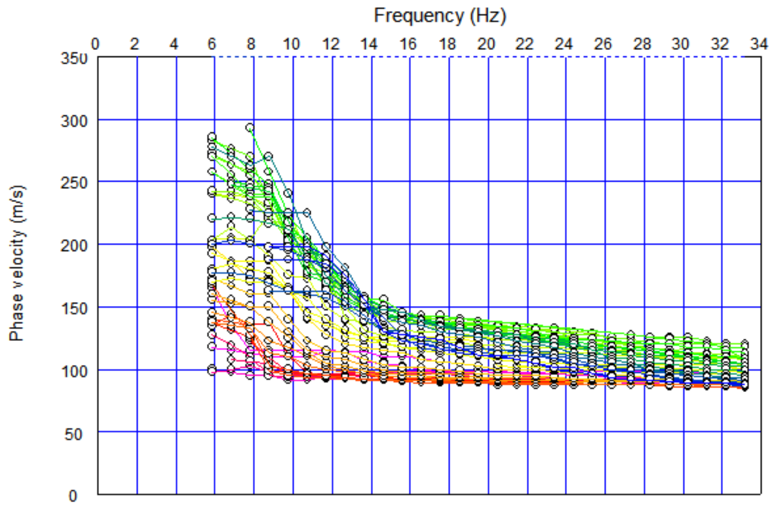


Figure 2.9 Example of dispersion curves. Difference of color indicates the difference of CMPCC location. Red to yellow curves are placed in beginning of the survey line and green to blue curves are placed in ending of the survey line.

4) Initial model

Initial models for inversion are created by a simple wavelength transformation (Xia et al., 1999) in which wavelength calculated from phase velocity and frequency is divided by three and plotted at depth.

5) Inversion

Inversion is applied to each dispersion curve with horizontal regularization. A non-linear least squares method and/or a Genetic Algorithm is used for optimizing the model. The number of layers is fixed as 15 and only S-wave velocities are changed throughout the inversion. P-wave velocities and densities are automatically changed based on empirical relationships (Kitsunezaki at al., 1999; Ludwig et al., 1970). Theoretical dispersion curves are calculated by a matrix method (Saito and Kabasawa, 1993). The theoretical phase velocity (effective mode) was defined as phase velocity whose amplitude is maximum at each frequency based on the medium response. The fundamental mode and higher modes (up to the 5^{th} mode) are taken into account. The inversion was performed based on minimization of differences between the observed and the effective mode phase velocities. The iterative process changes the S-wave velocities until a good fit is obtained between the observed and calculated phase velocities. We obtain 2D S-wave velocity structure by aligning the 1D S-wave velocity profiles. This resulted in the S-wave velocity structure along each line of the survey. Figures 2.10 and 2.11 show examples of 1D and 2D S-wave velocity profiles obtained by the inversion.

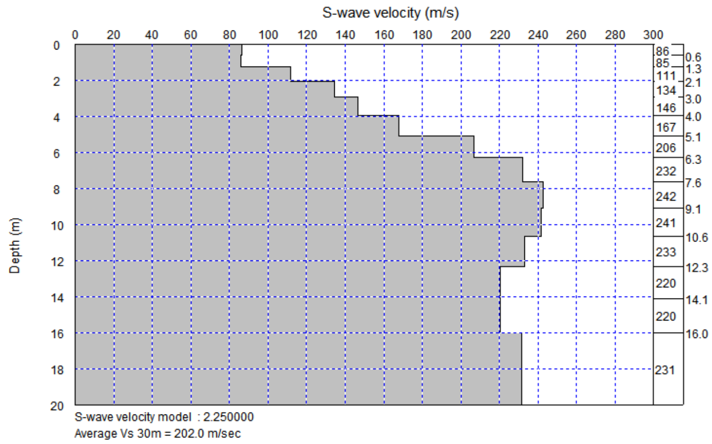


Figure 2.10 Example o1D S-wave velocity profile.

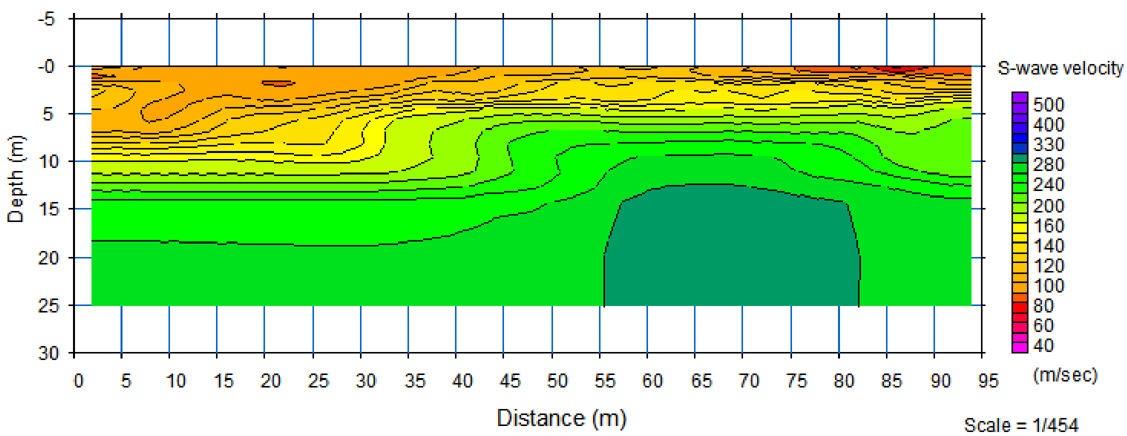


Figure 2.11 Example of 2D S-wave velocity model.

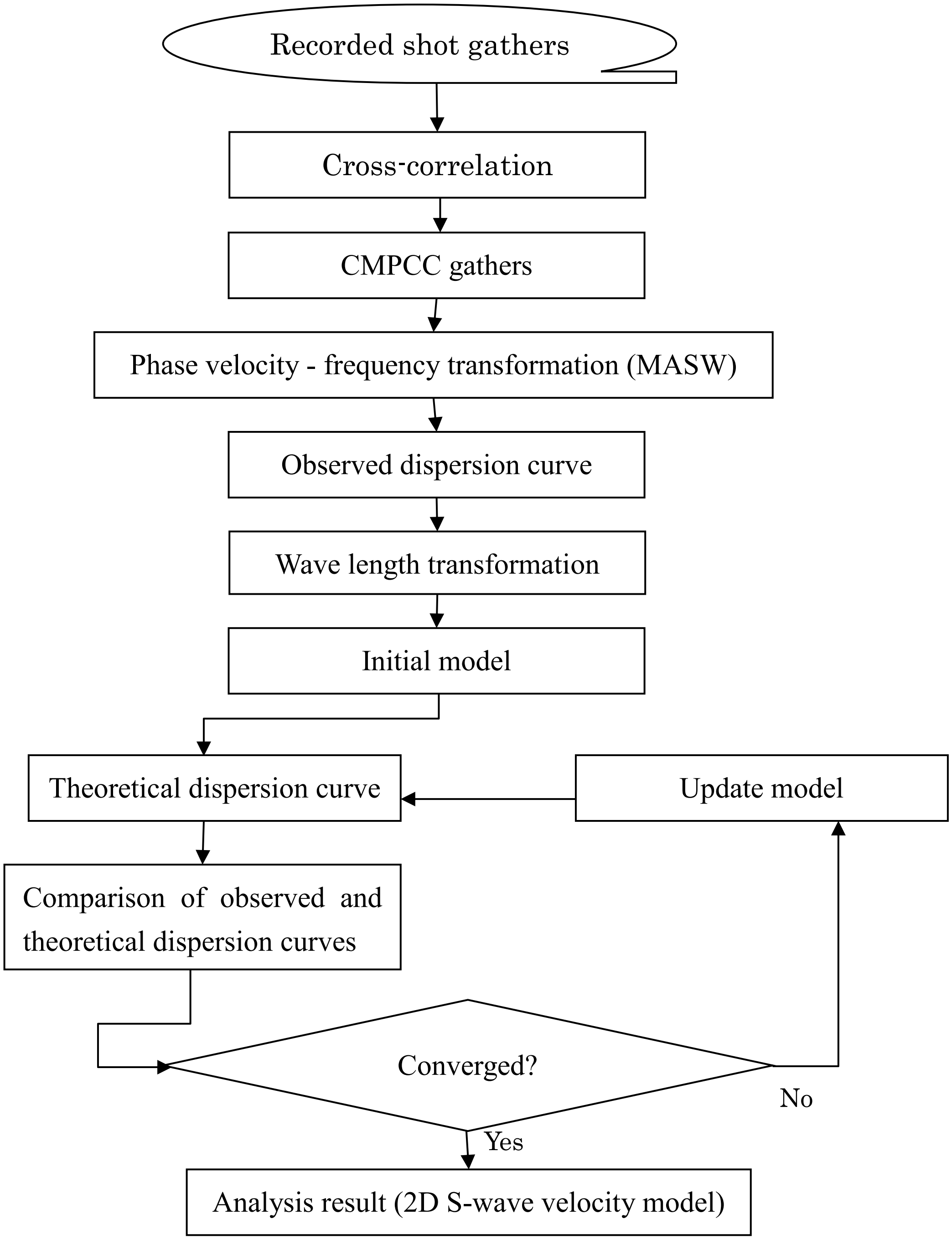


Figure 2.5 Processing flow of MASW.

3. Microtremor array measurements

3.1 Methodology

As mentioned in previous section, the surface wave method using active sources, such as a sledgehammer or a weight drop, have been applied to the delineation of shallow S-wave velocity structures. However, it is difficult to generate low-frequency surface waves using active sources. So delineating S-wave velocity structures deeper than a depth of 20 m (60 feet) is unreliable with active surface sources alone. In order to solve this problem, passive surface wave methods using microtremors (ambient noise) have been developed.

The earth's surface is always vibrating faintly. These vibrations are called ambient noises or microtremors. The microtremors are generated by various sources, such as wind, ocean waves at the seashore, traffic noises, heavy machinery in factories and household appliances. Because microtremors are generated by sources on the ground surface, the microtremors mainly consist of surface waves, and the vertical motion of the microtremors can be considered as Rayleigh waves. Therefore, it is reasonable that the dispersion curve of the vertical motion of the microtremors is the dispersion curve of Rayleigh waves and the sub-surface S-wave velocity structure can be estimated by analyzing dispersion characteristics of the microtremors. Figure 3.1 shows a schematic diagram of the surface wave method using microtremors. We will use the term "microtremor array measurements" to refer to the surface wave method using microtremors or ambient noises.

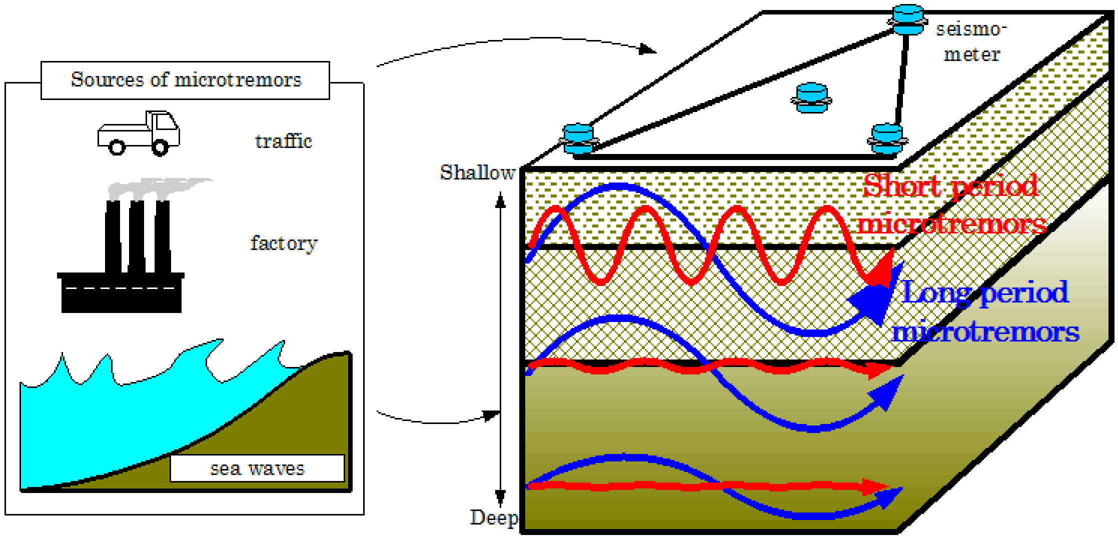


Figure 3.1 Schematic diagram of microtremor array measurements.

3.2 Instrumentation

Ten seismographs (Atom) manufactured by Geometrics and 10 vertical component geophones with 2 Hz of natural frequency manufactured by Sunfull were used for data acquisition. The seismographs include a GPS clock so that all seismographs can be synchronized to any distance without the use of cables. Figure 3.2 shows the Atom and geophone. Specifications for the equipment are shown in Table 3.1.

Figure 3.2 Seismograph Atom (right) and geophone (left) used in data acquisition.

Table 3.1 Specification of equipment

Equipment	Quantity	Manufacturer	Specification	
Geophone	10	Sunfull	Natural frequency	2 Hz
			Component	Vertical
Seismograph (Atom)	10	Geometrics	Preamp gain	0db, 12db, 24db, 36db
			Distortion	< 0.0001 %
			Data storage	4GB
			Frequency response	0.2 – 200Hz
			Dynamic range	124 dB (measured)
			A/D converter	24 bits
			Sampling time	0.25, 0.5, 1, 2, 4 msec
			Dimension	142×140×102 mm
			Weight	Approx. 1.6 kg

3.3 Data acquisition

Table 3.2 summarizes the shapes and sizes of the array used in the measurements. Ten pairs of seismographs and geophones were used. The vertical component of microtremors were recorded with a 4 msec sampling interval. Data lengths were 34 to 43 minutes at shallow arrays and 86 minutes for a deep array. Data acquisition for all measurements took one night. Figure 3.3 and 3.4 show the array configuration of shallow arrays.

Table 3.2 Shape and size of array used in the measurements.

Measurements	Array	Array shape	Array size (feet)	Sensor spacing (feet)	Number of sensors	Data length (min.)
Shallow arrays	G	L-shaped	300	25.0 to 459.6	10	43
	H	L-shaped	300	25.0 to 424.3	10	34
	I	L-shaped	300	25.0 to 424.3	10	35
Deep array	J	Random	1593	230.5 to 1592.6	10	86
Total		-	-	-	40	198

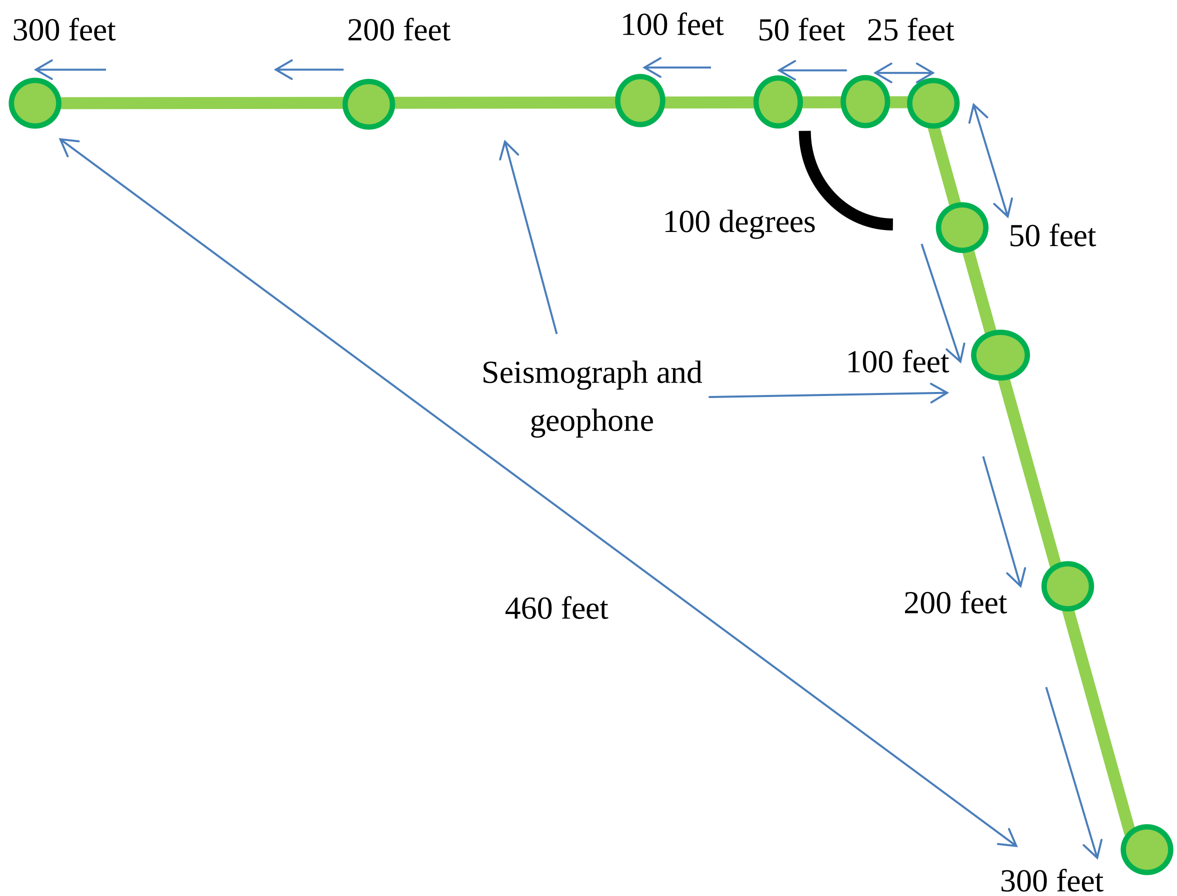


Figure 3.3 Array configurations (G array).

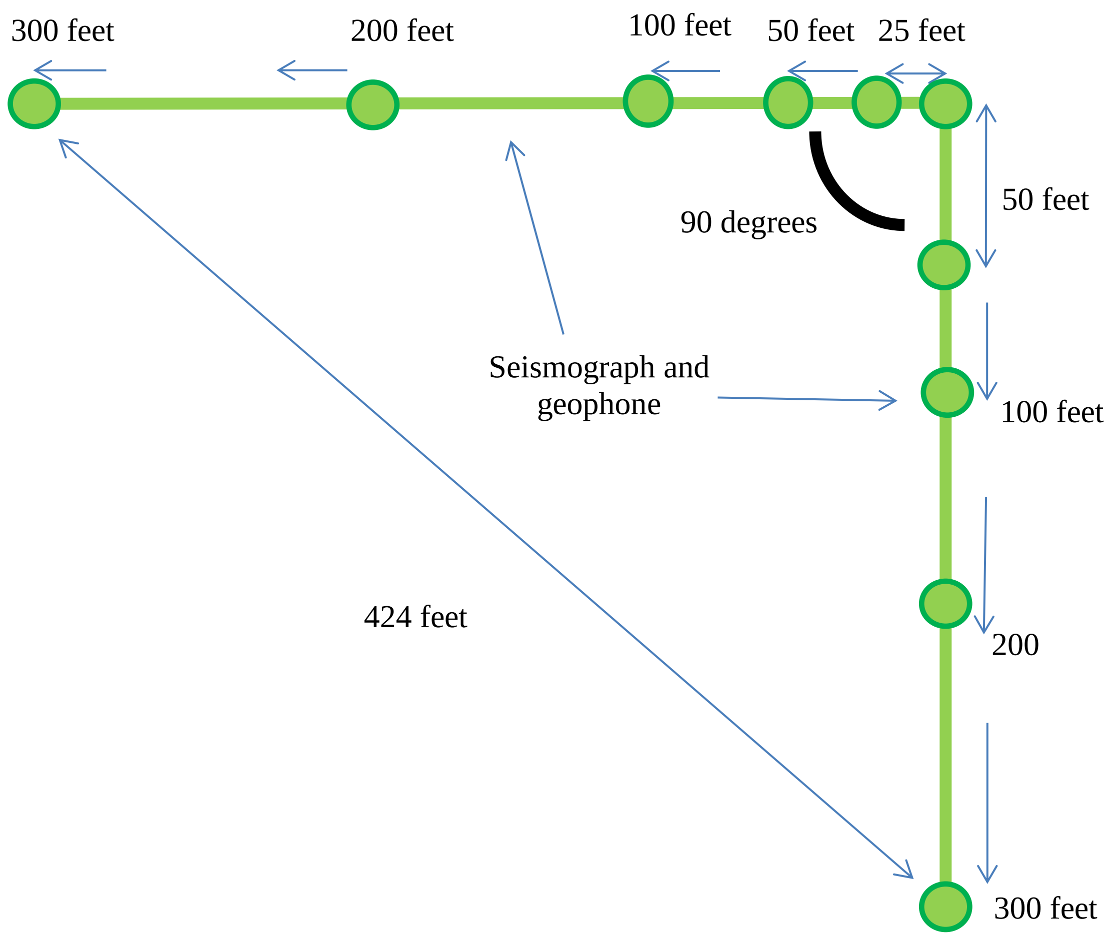


Figure 3.4 Array configurations (H and I arrays).

3.4 Analysis

Recorded microtremor data was analyzed using the Spatial Autocorrelation (SPAC) method. Data processing is summarized as follows and its flow is shown in Figure 3.5.

1) Pre-processing

The recorded vertical component of microtremor data is divided into several time blocks that are overlapped. Each block consists of 16,384 samples for a total length of 65.536 seconds. An example of a microtremor data block is shown in Figure 3.6. Several blocks including non-stationary noises such as moving vehicles were rejected and not processed.

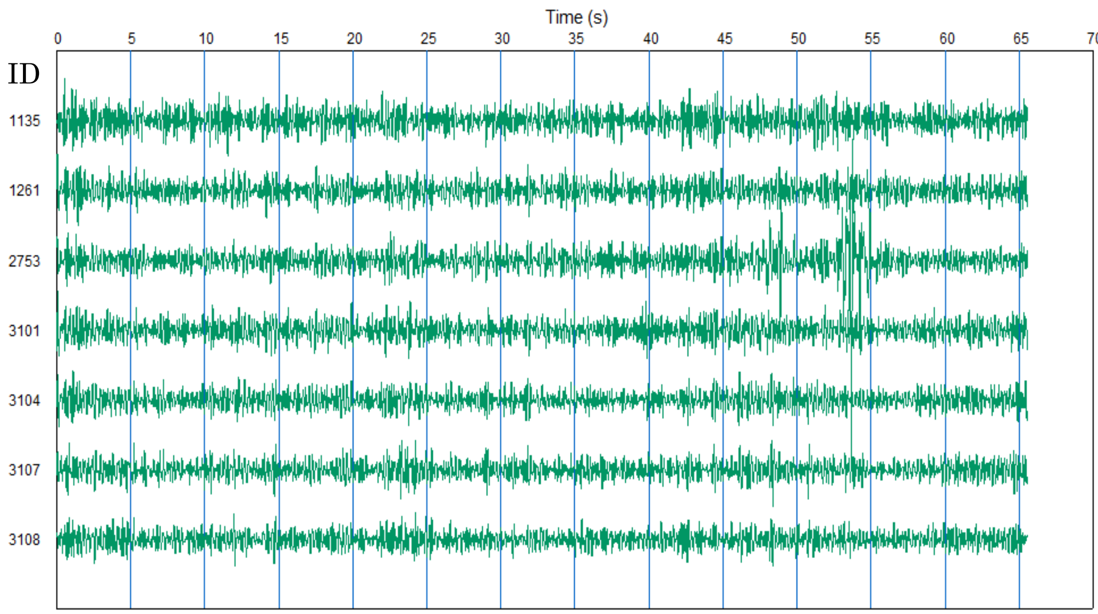


Figure 3.6 Example of microtremor data.

2) Spatial autocorrelation

A Fast Fourier Transform (FFT) is applied to each block to transform the time domain waveform data to the frequency domain. Coherence was first calculated for each block and then coherences of all blocks were averaged. The coherence (*COH*) is calculated by equation 3.1 shown below.

$$COH(\phi, r, \omega) = \frac{CC(\phi, r, \omega)}{AC(x, y, \omega) AC(x + \Delta x, y + \Delta y, \omega)} \quad (3.1)$$

$$\Delta x = r\cos\phi$$
$$\Delta y = r\sin\phi$$

Where, *x and y* are locations of seismographs, *r* is the distance between two seismographs, ω is the angular frequency, *CC* and *AC* are the cross-correlation and autocorrelation of microtremors recorded by the two seismographs respectively. Ten to one hundred blocks are averaged for calculating the final coherences. If the coherences are averaged over many blocks or over a long time, it can be considered as SPAC and expressed by Bessel function as shown by equation 3.2. Figure 3.7 shows an example of SPAC.

$$\int_{\phi=0}^{\phi=2\pi} COH(\phi, r, \omega)d\phi = J_0\left(\frac{\omega}{c(\omega)}r\right) \qquad (3.2)$$

Where $c(\omega)$ is the phase velocity of microtremors, J_0 is the first kind of Bessel function.

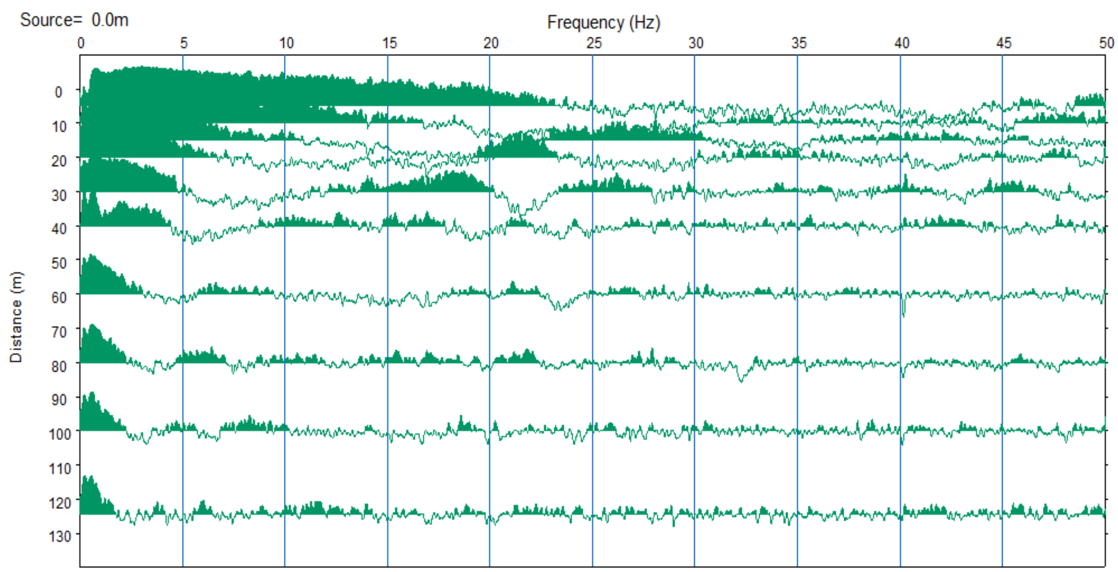


Figure 3.7 Example of spatial autocorrelations.

A phase velocity can be determined at each frequency so that the difference between both sides of equation 3.2 are minimized. This series of phase velocities defines a dispersion curve. Figure 3.8 shows an example of a dispersion curve.

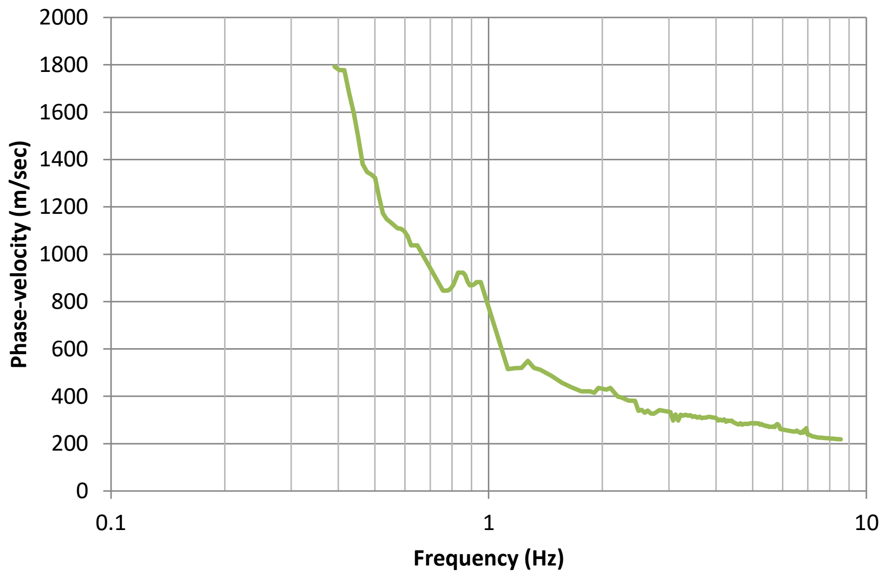


Figure 3.8 Example of a dispersion curve.

3) Initial model

Initial models for inversions are created by a simple wavelength transformation (Xia et al., 1999) in which wavelength is calculated from phase velocity and frequency is divided by three and plotted at depth.

4) Inversion

A non-linear least squares method and/or a Genetic Algorithm is used for optimizing the model. The number of layers is fixed as 19 and only S-wave velocities are changed throughout the inversion. Density is automatically changed based on empirical relationships (Ludwig et al., 1970). P-wave velocity above ground water level is the double of the S-wave velocity and beneath the ground water level is automatically changed based on empirical relationship (Kitsunezaki at al., 1999). Ground water level is set to a depth of 22.9, 6.7 and 0.0 feet at G-, H- and I-arrays respectively based on existed drilling information at the investigation site. Theoretical dispersion curves are calculated by a matrix method (Saito and Kabasawa, 1993). Fundamental mode and higher modes up to 5th mode were taken into account in the calculation of theoretical dispersion curves. The iterative process changes the S-wave velocities until a good fit is obtained between the observed and calculated phase velocities. Figure 3.9 shows an example of an inverted S-wave velocity model.

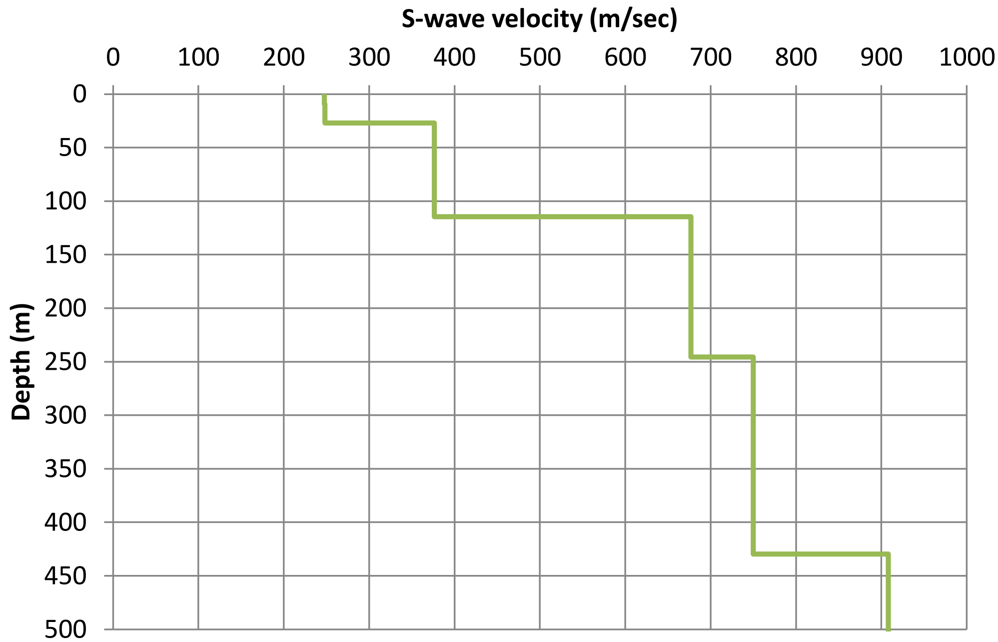


Figure 3.9 Example of inverted S-wave velocity model.

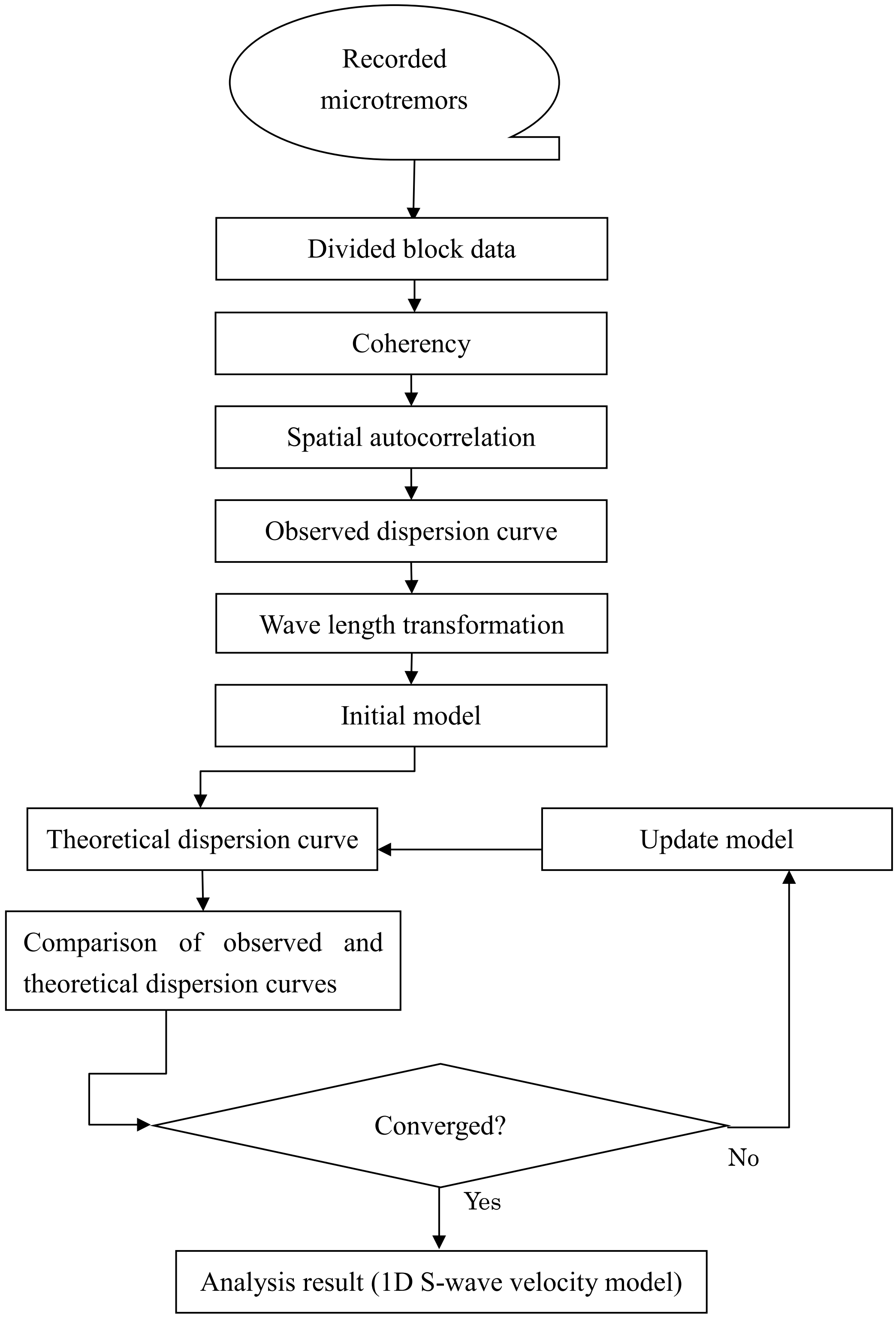


Figure 3.5 Processing flow.

4. Vibration monitoring

4.1 Methodology

The purpose of the vibration monitoring is the understanding of vibration characteristics at the site. Vibration at the ground surface consists of elastic or seismic waves that were used in the surface wave method and microtremor array measurements described in previous sections. Four three-component broadband accelerometers were used in the vibration monitoring to measure elastic waves at the ground surface. The monitoring focuses on following three issues.

A) Attenuation characteristics of vibration with distance
Any elastic waves attenuate with distance. Attenuation characteristics depend on site condition, frequency of interests, etc. To figure out the attenuation characteristics, four accelerometers were placed on a K-line from a speed bump on a road at the northern entrance (Figure 1.6). A trailer and an SUV passed on the speed bump several times to generate elastic waves.

B) Difference of vibration level associated with traffic condition
It seems that most vibrations or elastic waves are generated by traffic or construction noises at the site. It appears that the vibration level changes associated with traffic or construction. To figure out the change of vibration level associated with the traffic or construction noises vibration was measured at different time periods on the same line (L-line: Figure 1.7). Measurements were carried out during a busy traffic period (morning) and a low volume traffic period (midnight).

C) Difference of vibration level depending on sites
Vibration level generally changes associated with geological conditions. To figure out the difference of vibration levels depending on sites, the vibration was measured at two sites (L-line and M-line : Figure 1.8) and compared.

Table 4.1 summarizes the specification and measurement periods of the vibration monitoring.

Table 4.1 Specification and measurement periods of the vibration monitoring

Array name	Site	Number of accelerometers	Array length (feet)	Measurement purpose	Measurements time	Data length (min.)
K	Parking lot	4	149	Vibration from a speed bump	7:15 am to 7:30 am	15
L	North entrance	4	300	Noisy and quiet periods	6:10 am to 7:10 am	60
					7:37 am to 10:00 am	143
					9:10 pm to 1:02 am	232
M	South entrance	4	300	Noisy period	3:00 pm to 7:00 am	240
Total	-	12	749	-		

4.2 Instrumentation

Four three-component broadband seismographs (McSEIS-MT Neo) manufactured by OYO Corporation were used for data acquisition. Sensors are accelerometers and three component accelerations on the ground surface were measured. The seismographs include a GPS clock so that two seismographs could be synchronized at any distance. Figure 4.1 shows the McSEIS-MT Neo and its main specifications are shown in Table 4.2.

Figure 4.1 Seismographs (McSEIS MT-Neo) used in vibration monitoring.

Table 4.2 Specification of McSEIS MT-Neo

Sensor	Servo-accelerometer
Resolution	1μG
Sensitivity	2.0V/G
Range	+/-4G
Frequency Response	0.1 – 200Hz
Dynamic range	120dB (Measured)
A/D Converter	32bits
Sampling time	2, 4, 10, 20, 50msec
Dimension	220mm(W) ×245mm(D) ×250mm (H)
Weight:	Approx. 7.5kg (Main unit 5kg, internal battery 2.5kg)

4.3 Data acquisition

Four seismographs were deployed at 50 feet spaced intervals on the L-line and at 100 feet spaced intervals on the L-and M-lines and three-component accelerations on the ground surface were continuously recorded with a 4 msec sampling interval. Major vibration events, such as a tracked vehicle or a bus passed the sites, were noted with the time.

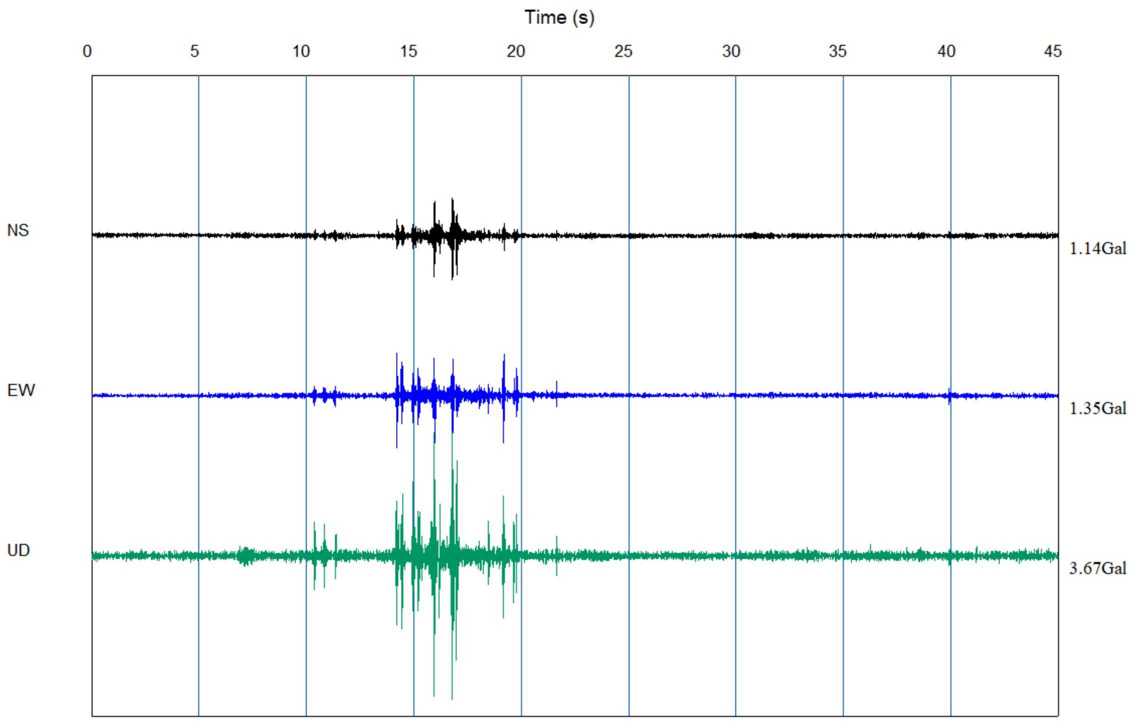
4.4 Analysis

Vibration levels were evaluated in terms of an octave band analysis to investigate vibration characteristics depending on frequency. The octave analysis can be summarized as follows.

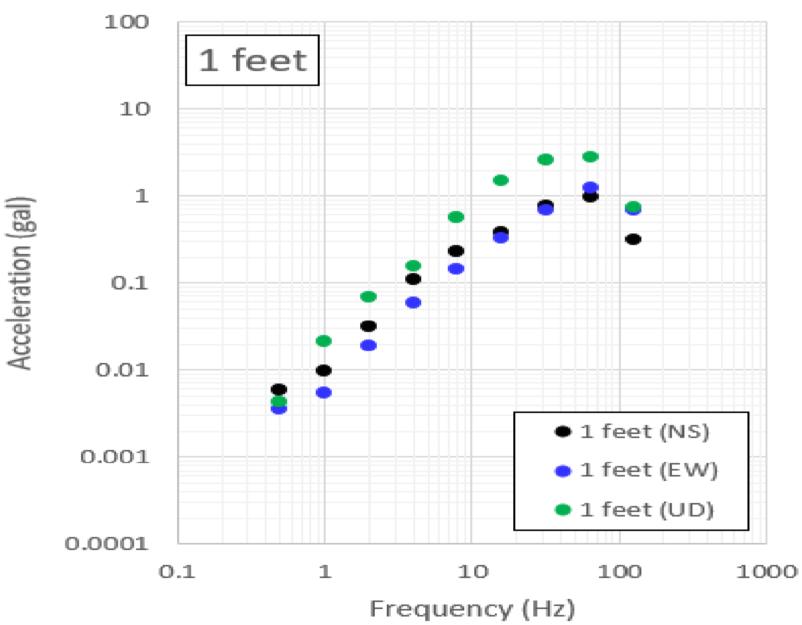
1) Nine center frequencies were chosen as the octave sequence.
2) Low-cut and high-cut frequencies were calculated by dividing or multiplying the center frequencies by $\sqrt{2}$. Table 4.3 shows the center, low-cut and high-cut frequencies.
3) For each center frequency, a bandpass filter with the low-cut and high-cut frequencies is applied and the maximum amplitude was picked in the time domain.
4) Selected maximum amplitudes are plotted in a frequency-amplitude chart. Figure 4.2 shows an example of raw waveform data and the result of the octave analysis. The maximum amplitude in the octave analysis is generally consistent with the maximum amplitude in raw waveform data.

Table 4.3 Bandwidths used in an octave analysis.

Center frequency (Hz)	Low-cut frequency (Hz)	High-cut frequency (Hz)
0.5	0.353553	0.707107
1	0.707107	1.414214
2	1.414214	2.828427
4	2.828427	5.656854
8	5.656854	11.31371
16	11.31371	22.62742
32	22.62742	45.25483
64	45.25483	90.50967
128	90.50967	181.0193



a) Raw data in time domain.



b) Result of octave analysis.

Figure 4.2 Example of raw waveform data and the result of the octave analysis.

5. Investigation results

5.1 2D high resolution surface wave method (MASW)

1) Shot gathers

The example of raw shot gather obtained by a 2D high resolution surface wave method (multi-channel analysis of surface waves: MASW) is shown in Figure 5.1. A 20 lb. sledge hammer was used as an energy source. We can see that clear surface waves were observed.

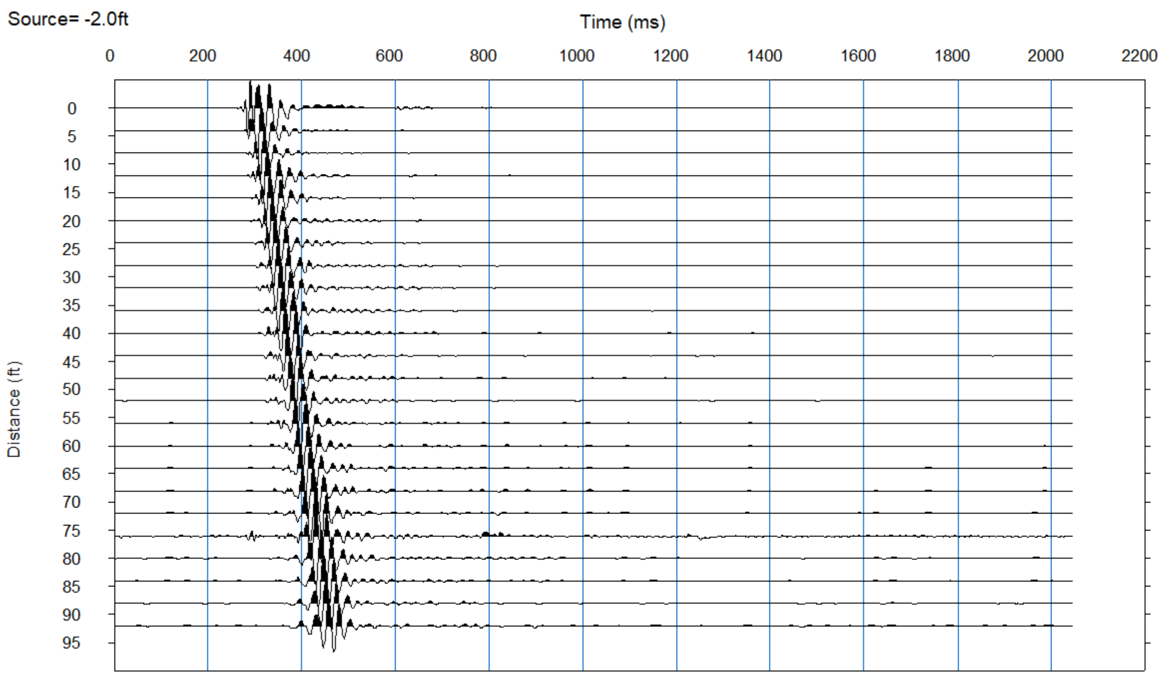


Figure 5.1 An example of raw shot gathers obtained by a multi-channel analysis of surface waves (MASW).

2) Phase velocity-frequency images

The example of phase velocity image in frequency domain is shown in Figure 5.2. The image was calculated from a CMPCC gather and not from a raw shot gather. The CMPCC gathers were calculated with 8 foot intervals. In the figure, the difference of color indicates difference in amplitude. The blue color indicates a large amplitude and the red color indicates a small amplitude. The maximum amplitude phase velocity is selected at each frequency and a series of phase velocities result in a dispersion curve. We can see a clear dispersion curve in a frequency range between 20 and 100 Hz.

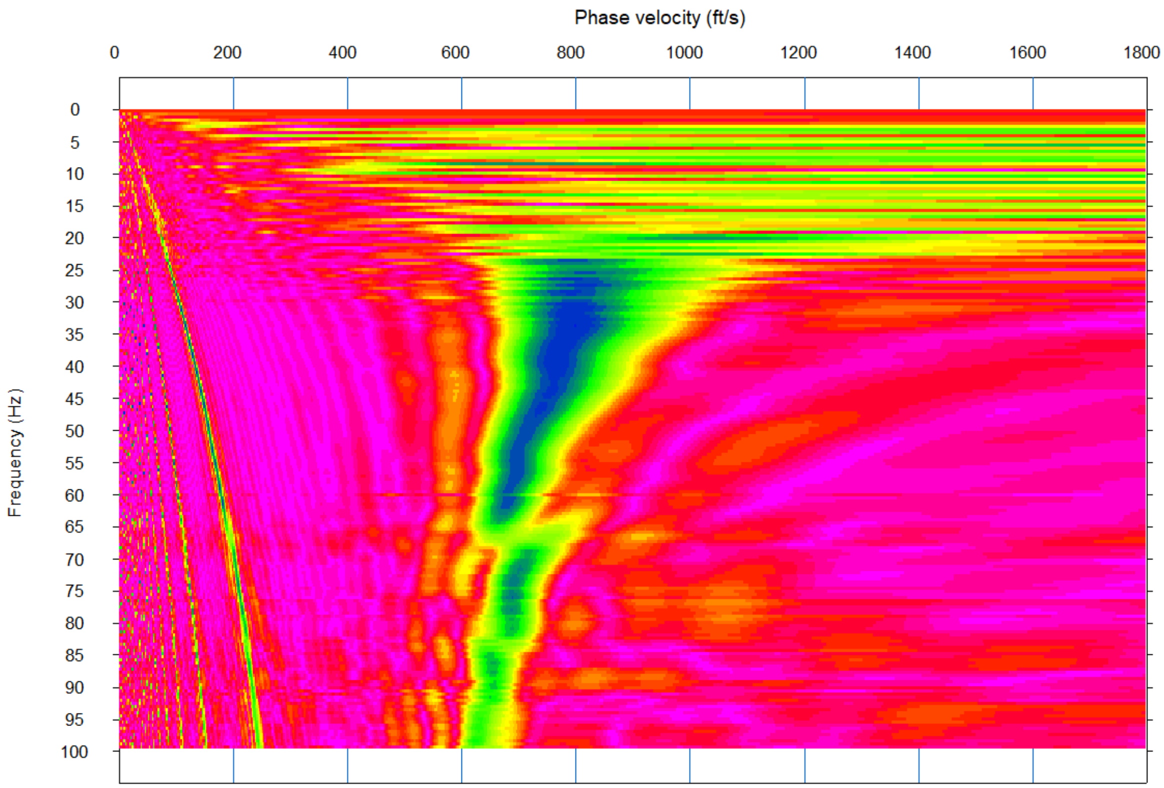


Figure 5.2 An example of phase velocity image in frequency domain.

3) Dispersion curves

Figures 5.3 to 5. 8 show dispersion curves of each line. In the figures, the difference of color indicates the difference of CMPCC location. Red to yellow curves are placed at the beginning of the survey line and green to blue curves are placed at the ending of the survey line.

There is no clear difference of dispersion curves in each line. It implies that S-wave velocities in each line do not have large horizontal change. Figure 5.9 compares dispersion curves at the middle of each line. We can see clear difference among lines. Phase velocity is generally low at A-, B- and D-lines and high at C-, E- and F-lines. It implies that the S-wave velocity of A-, B- and D-lines are generally high compared with C-, E- and F-lines.

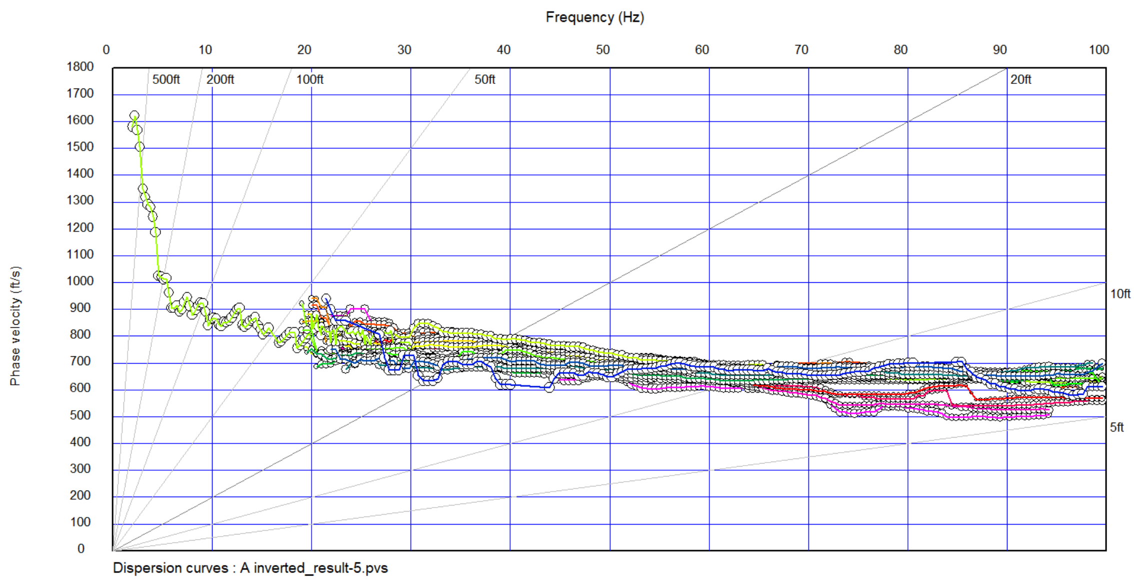


Figure 5.3 Dispersion curves (A-line).

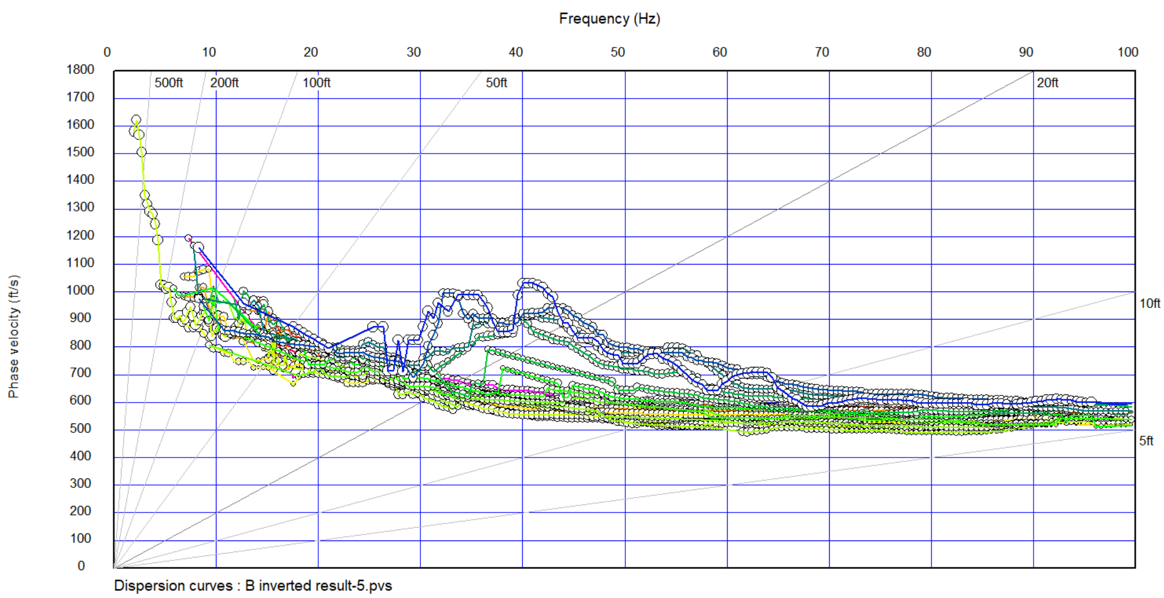


Figure 5.4 Dispersion curves (B-line).

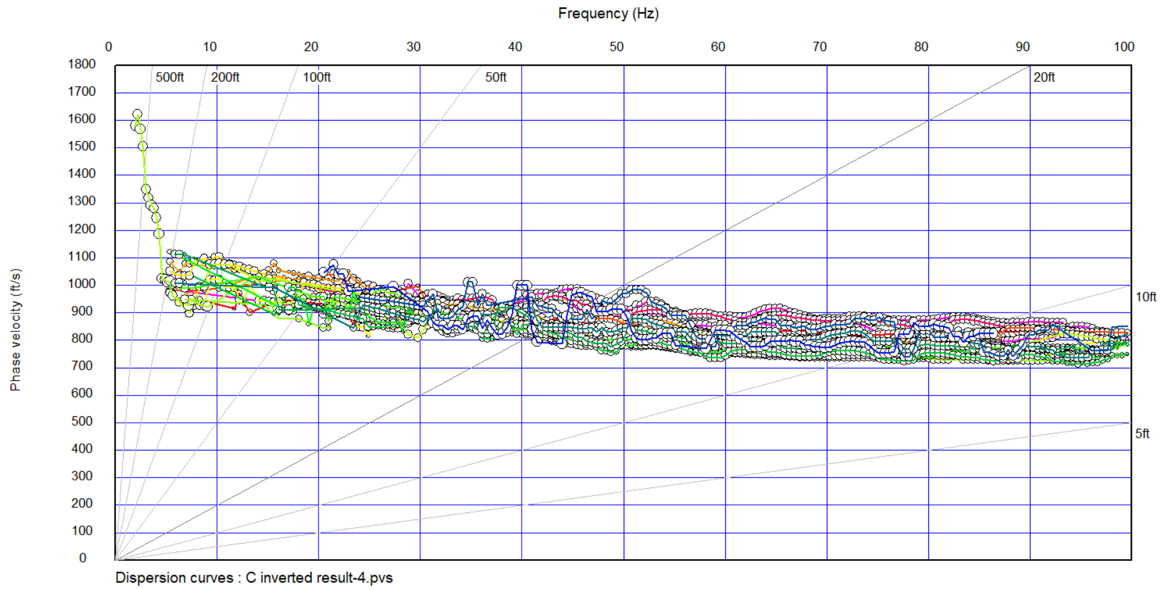


Figure 5.5 Dispersion curves (C-line).

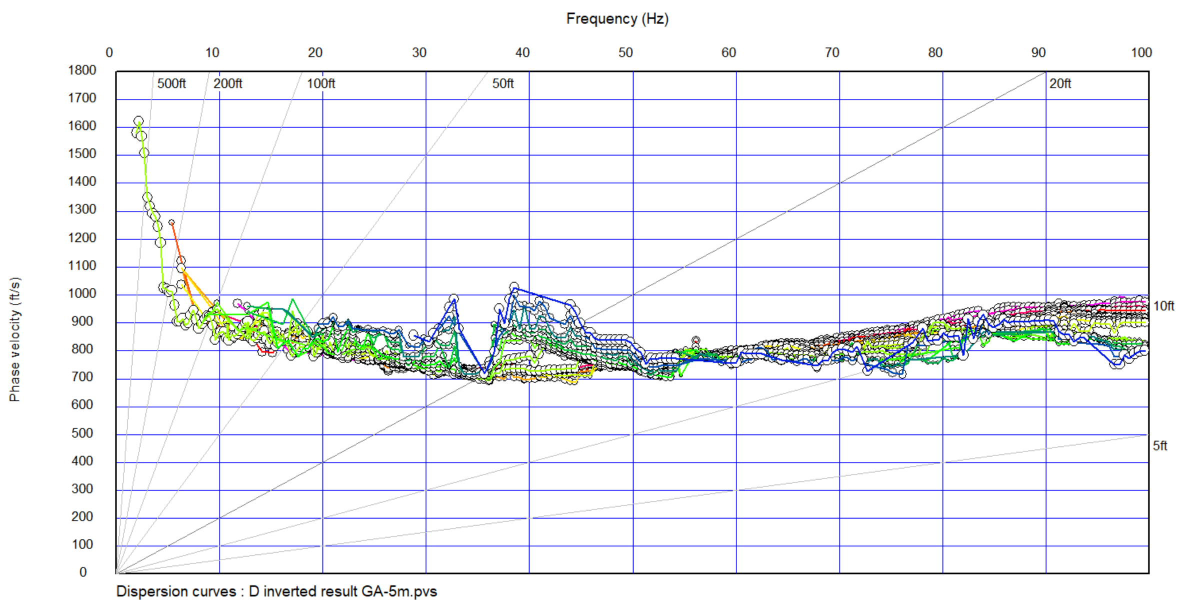


Figure 5.6 Dispersion curves (D-line).

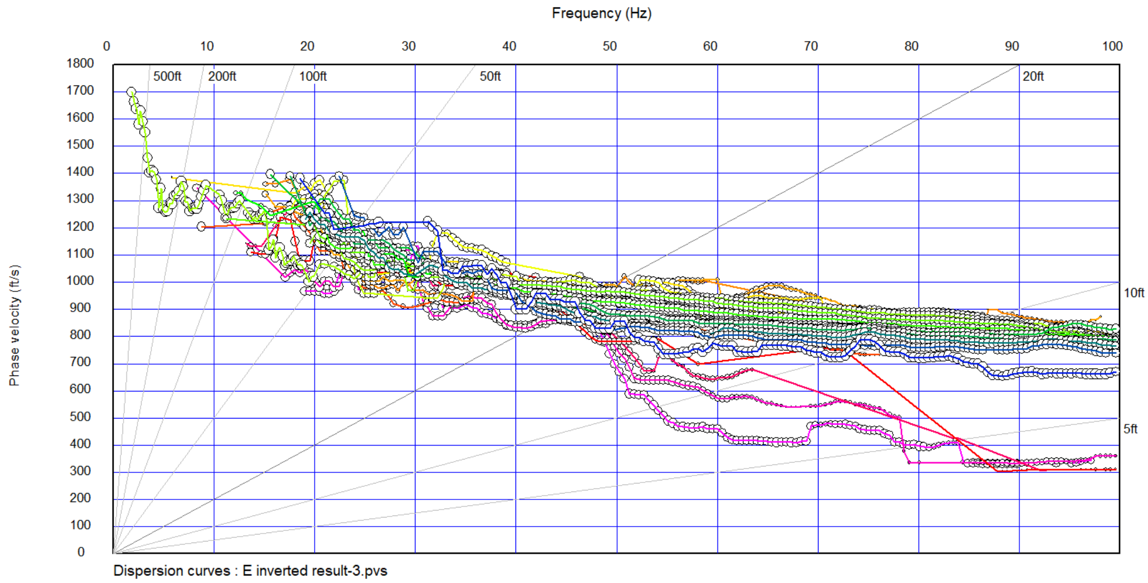


Figure 5.7 Dispersion curves (E-line).

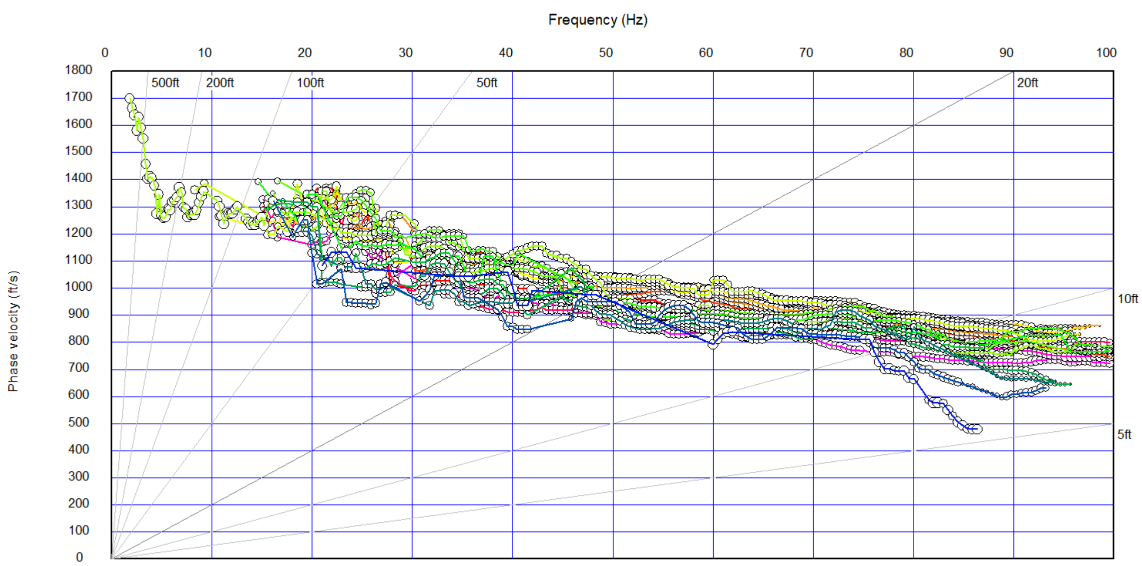


Figure 5.8 Dispersion curves (F-line).

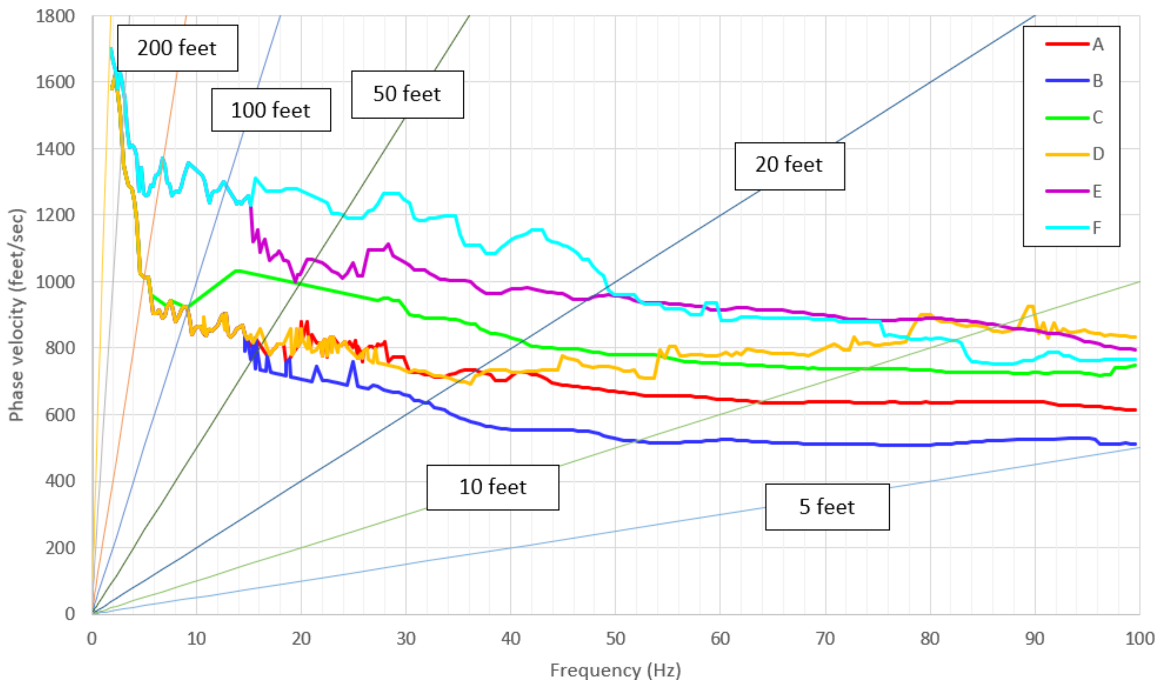


Figure 5.9 Comparison of dispersion curves.

4) Analysis result

Figures 5.10 to 5.15 show analyzed 2D S-save velocity cross sections. There is no significant horizontal velocity change at shallow depths in each line. S-wave velocity is generally higher at C-, E- and F-lines compared with other lines in a depth deeper than 10 feet. Figure 5.16 compares 1D S-wave velocity profiles at the middle of each survey line. S-wave velocities are generally lower at A- and B-lines at shallow depth (< 10 feet). At greater depth (> 15 feet), S-wave velocities are higher at E- and F-lines.

Table 5.1 summarizes obtained 1D S-wave velocities, average S-wave velocity to a depth of 100 feet (AVS 100 feet), and the Site Class at the middle of each survey line. Note that the penetration depth of the surface wave method is approximately 30 feet in the investigations and the AVS 100 feet and the Site Class are just for reference purposes.

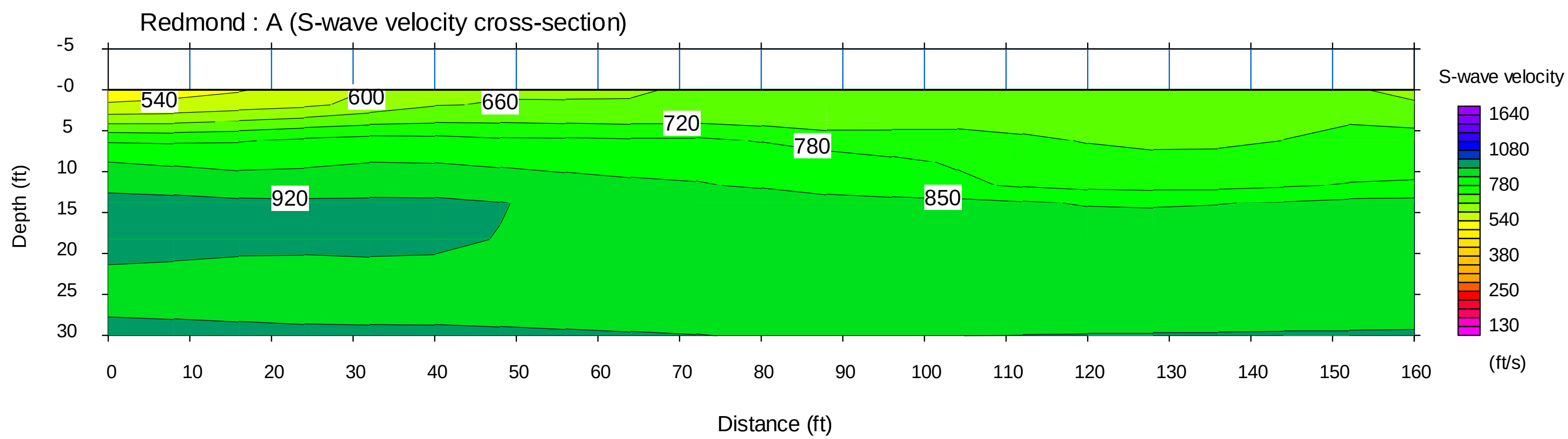


Figure 5.10 2D S-save velocity cross section obtained by surface wave method (A-line).

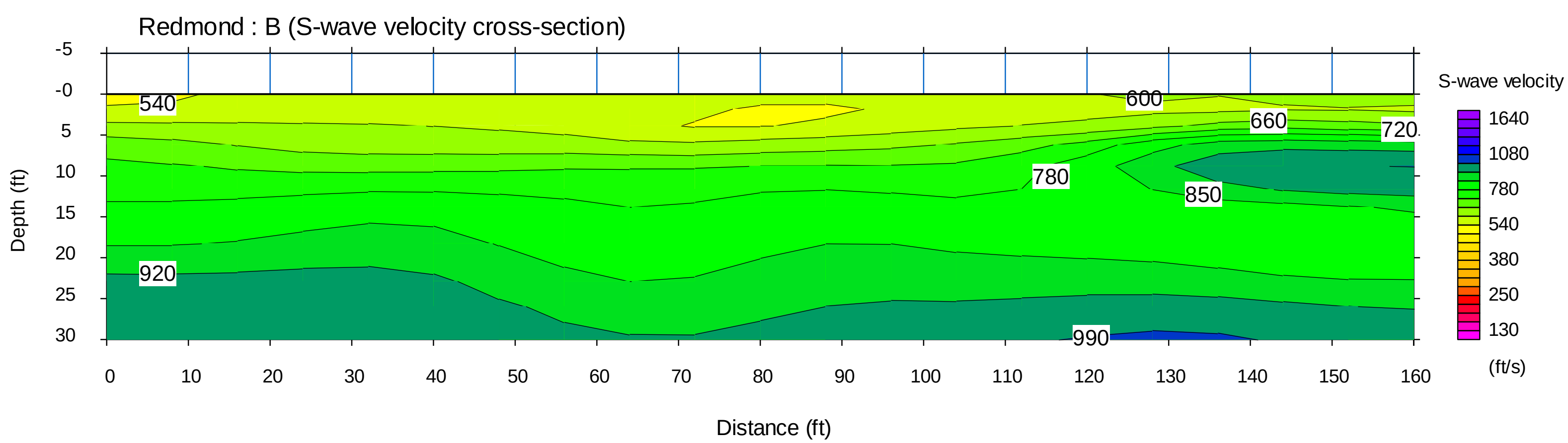


Figure 5.11 2D S-save velocity cross section obtained by surface wave method (B-line).

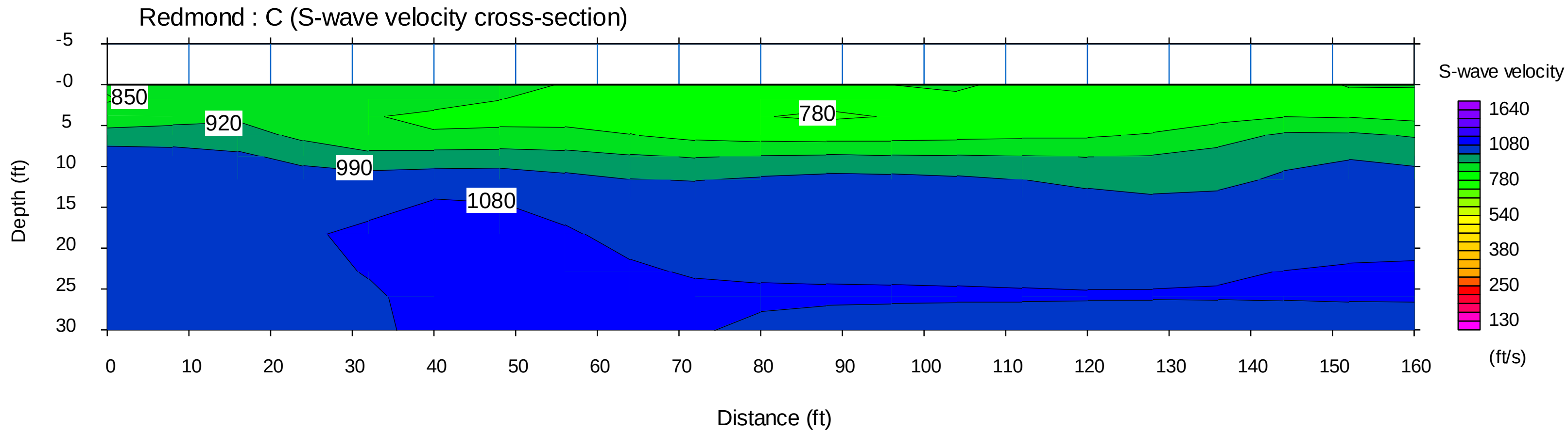


Figure 5.12 2D S-save velocity cross section obtained by surface wave method (C-line).

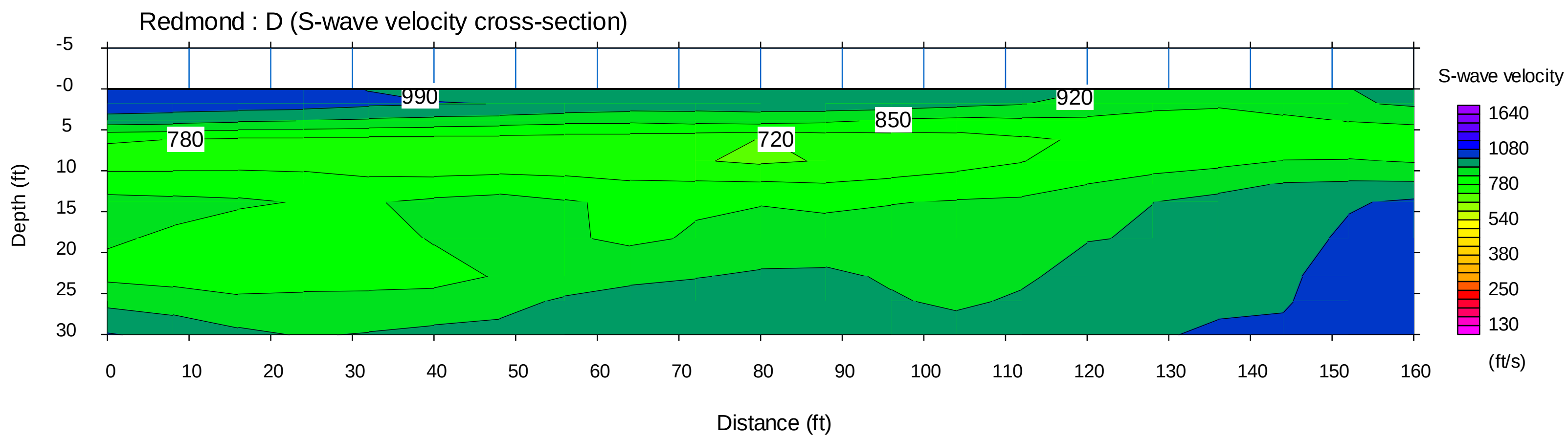


Figure 5.13 2D S-save velocity cross section obtained by surface wave method (D-line).

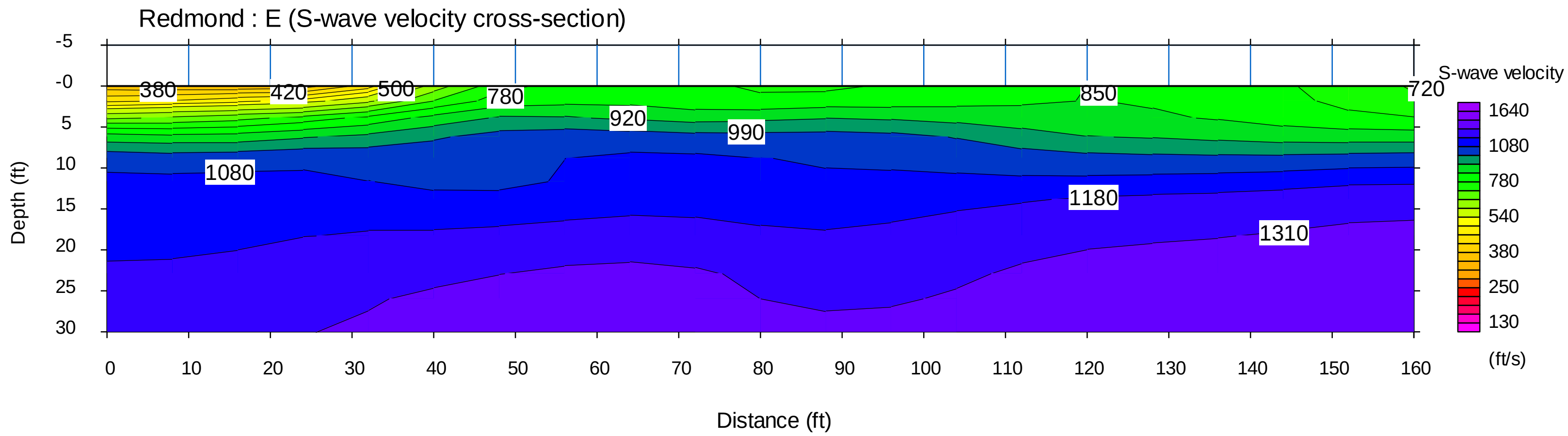


Figure 5.14 2D S-save velocity model obtained by surface wave method (E-line).

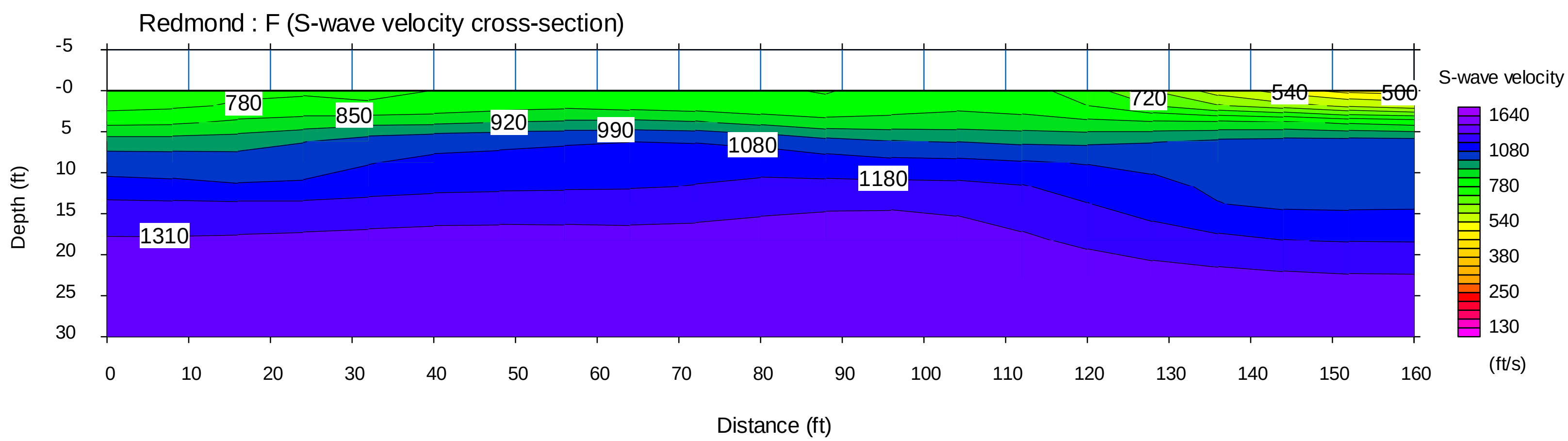


Figure 5.15 2D S-save velocity model obtained by surface wave method (F-line).

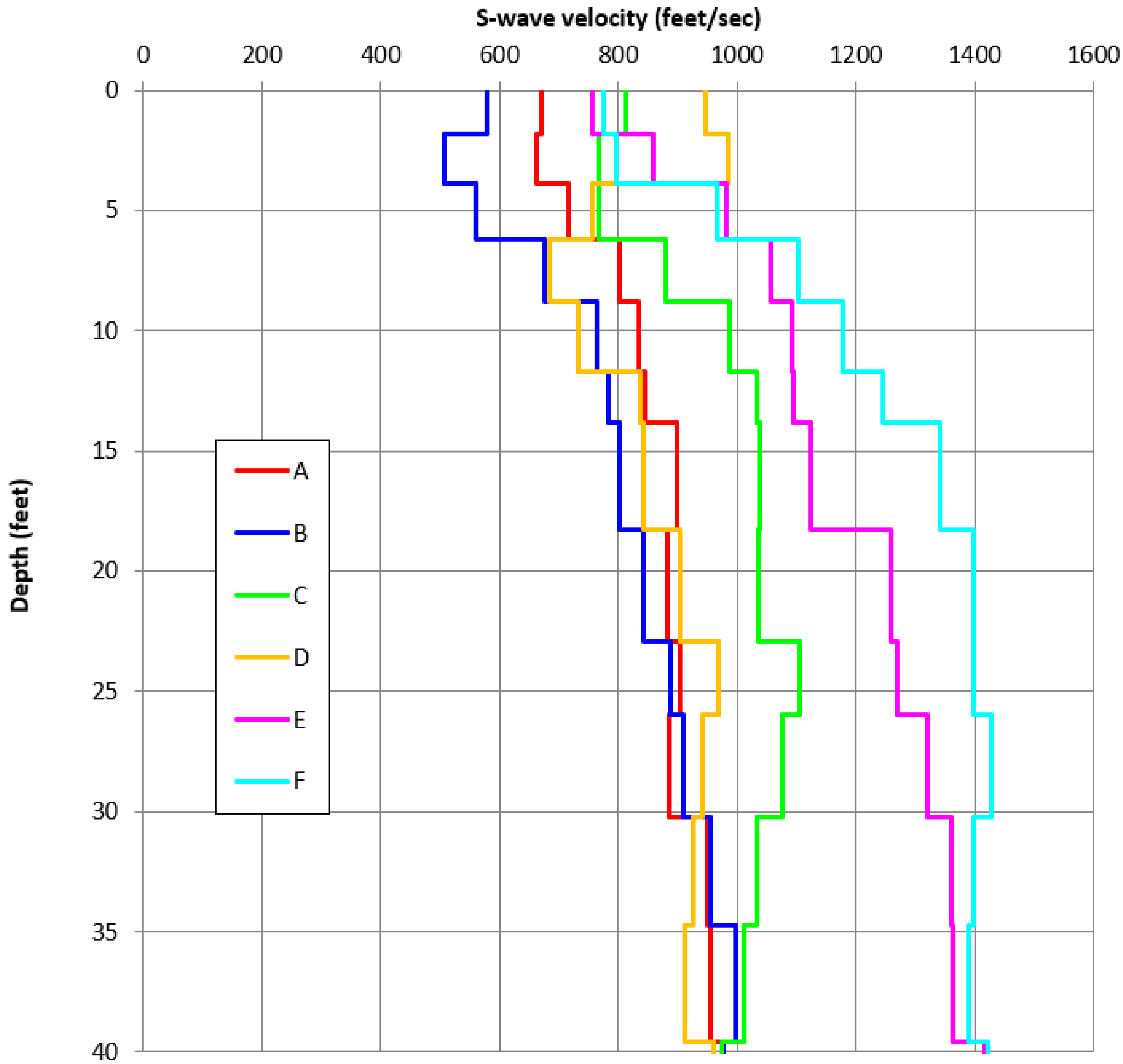


Figure 5.16 Comparison of 1D S-wave velocity profiles obtained by surface wave method (MASW) at a middle of each survey line.

Table 5.1 Comparison of shallow S-wave velocity profiles obtained by surface wave method (MASW) at a middle of each survey line.

Depth (feet)		S-wave velocity (feet/sec)					
top	bottom	A	B	C	D	E	F
0.0	1.8	669.7	580.0	813.5	947.6	757.0	775.0
1.8	3.8	662.2	506.4	768.2	983.9	859.0	796.0
3.8	6.2	714.8	559.4	767.0	754.9	982.3	965.0
6.2	8.8	803.1	674.4	878.7	682.4	1056.0	1102.4
8.8	11.7	834.5	764.7	986.6	732.0	1090.6	1178.5
11.7	13.8	844.0	782.7	1033.8	835.8	1093.5	1243.9
13.8	18.3	898.5	802.0	1038.0	840.9	1123.2	1340.3
18.3	22.9	882.7	841.9	1035.1	902.8	1259.7	1397.5
22.9	26.0	903.0	889.1	1106.6	968.3	1269.5	1398.8
26.0	30.2	886.1	909.7	1075.3	942.0	1321.4	1426.3
30.2	34.8	948.7	954.3	1032.0	926.3	1360.5	1397.6
34.8	39.6	954.3	997.3	1012.6	910.6	1363.5	1389.8
39.6	44.6	973.4	976.4	974.2	961.3	1416.1	1422.7
44.6	60.7	999.4	970.2	983.2	1014.7	1395.4	1419.0
60.7	-	1215.1	1337.3	1268.7	1255.0	1521.5	1510.0
AVS 100 feet (feet/sec)		998.7	984.3	1074.8	1019.3	1328.6	1364.7
Site class		D	D	D	D	C	C

5.2 Microtremor array measurements

1) Huddle test

In order to evaluate the minimum frequency that can be used for analysis, a test known as a huddle test was conducted. All seismographs are placed at the same location and simultaneous measurements are performed. The coherences between a pair of seismographs are then computed. Figure 5.17 shows the example of coherences calculated from microtremor data recorded in the huddle test. The coherence is generally larger than 0.9 in a frequency range between 0.4 and 40 Hz and it implies that phase velocities can be obtained in a frequency range between 0.4 and 40 Hz.

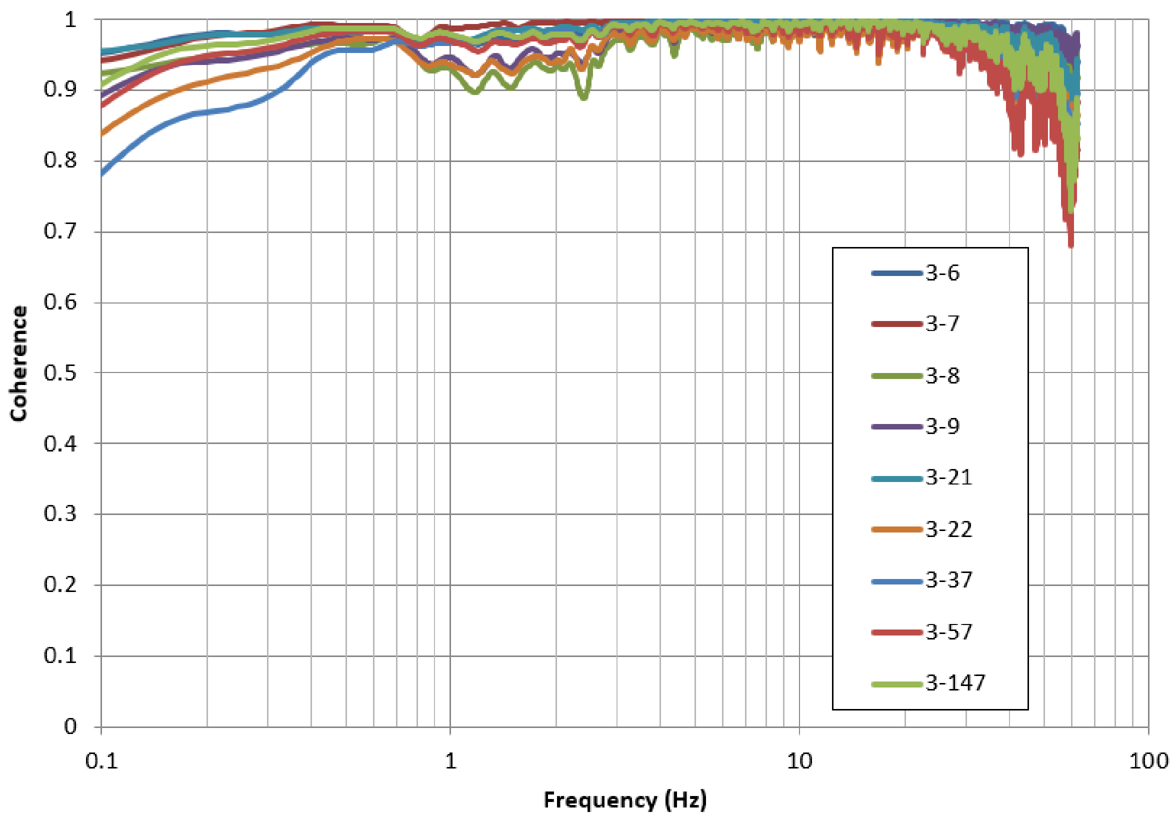


Figure 5.17 Example of coherences between two seismographs.

2) Amplitude spectra

Figure 5.18 shows an example of vertical component of amplitude spectra. The data was obtained from the J-array (deep array). The seismographs are geophones (velocity meter) with a natural frequency of 2 Hz and the amplitude of microtremors decreases in a frequency range lower than 2 Hz. There is a vague peak in the amplitude spectra at a frequency of 15 Hz.

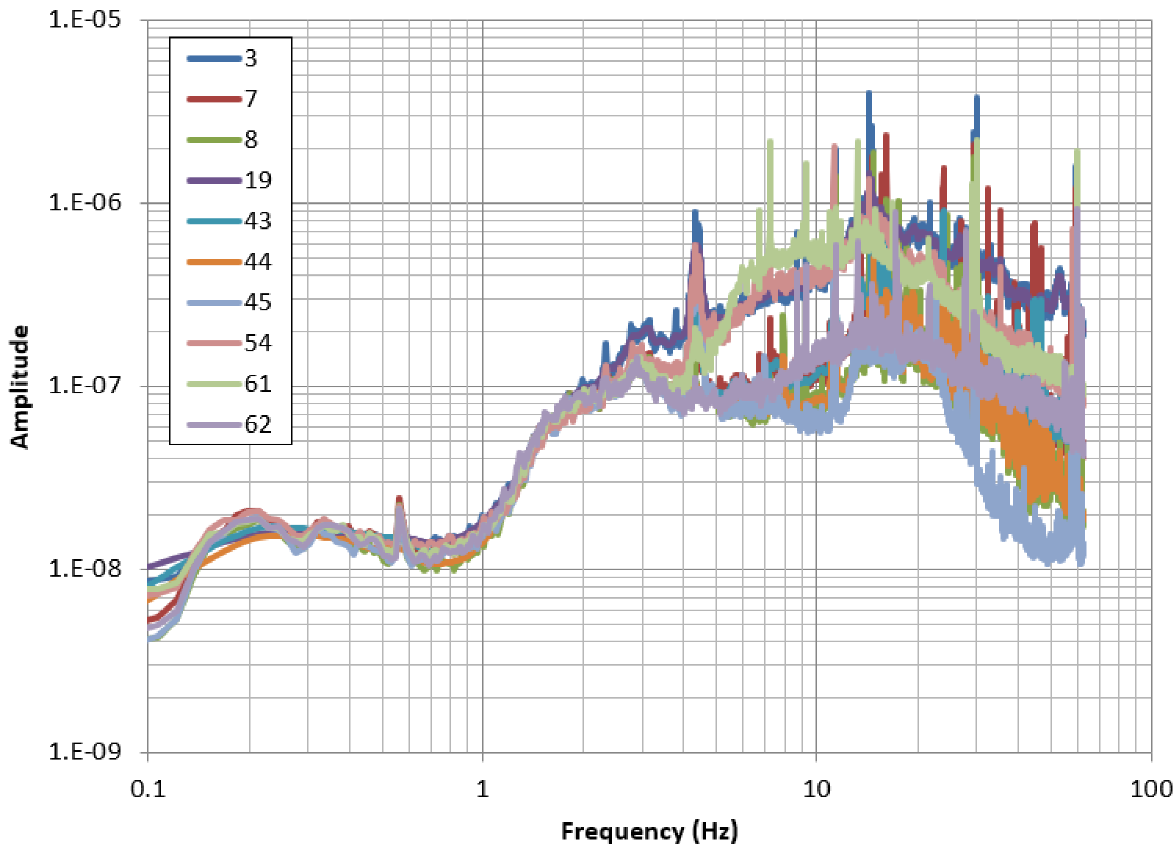


Figure 5.18 Example of amplitude spectra.

3) Spatial autocorrelation (SPAC)

Figures 5.19 to 5.21 show the spatial autocorrelations (SPAC) of shallow arrays and Figure 5.22 shows one of deep array. Coherences look like Bessel function in all arrays. Coherences in the deep array (Figure 5.22) decrease in the frequency lower than 0.2 Hz and it indicates that the phase velocity can be determined down to a frequency of 0.2 Hz.

There is a clear difference between coherences of the shallow arrays. A frequency of short distance (25 ~ 150 feet) coherences at the G-array is lower than that of the H- and I-arrays. For example, coherence with a distance of 50 feet is 0 at a frequency of 6 Hz at the G-array. On the contrary, it is 0 at a frequency of 10 Hz at the H- and I-arrays. It indicates that the high frequency phase velocities of the G-array are lower than one of the H-and I-arrays.

4) Dispersion curve

Figures 5.23 to 5.26 show phase velocity images in the frequency domain calculated from the SPAC. In the figure, differences of color indicate differences of error between observed coherences and theoretical Bessel functions shown in equation 3.2. Blue color indicates small error and red color indicates large error. Red dots show phase velocities that give minimum difference between both sides of equation 3.2. A dispersion curve is determined in a frequency ranges between 2.0 and 30 ~ 40 Hz at the shallow arrays and 0.4 to 5 Hz at the deep array. Like coherences mentioned before, there is clear difference among phase velocity images of shallow arrays G, H and I. At frequency range between 4 and 20 Hz, the phase velocities at the G-array are clearly lower than one of the H- and I-arrays. It indicates that the S-wave velocity at the array G is lower than one of H- and I-arrays in shallow depth.

Figure 5.27 shows the comparison of observed dispersion curves. In a frequency range between 4 to 20 Hz, the phase velocities of G-array are lower than one of the H- and I-arrays. In the frequency range between 2 and 4 Hz, the dispersion curves of shallow arrays (G, H and I) agree with the deep array (J). In the inversion, the dispersion curve of the deep array was concatenated to the other three dispersion curves of the shallow arrays. These three dispersion curves with a frequency range between 0.4 and 30 ~ 40 Hz were used in the inversion and three velocity models were analyzed for arrays G, H and I.

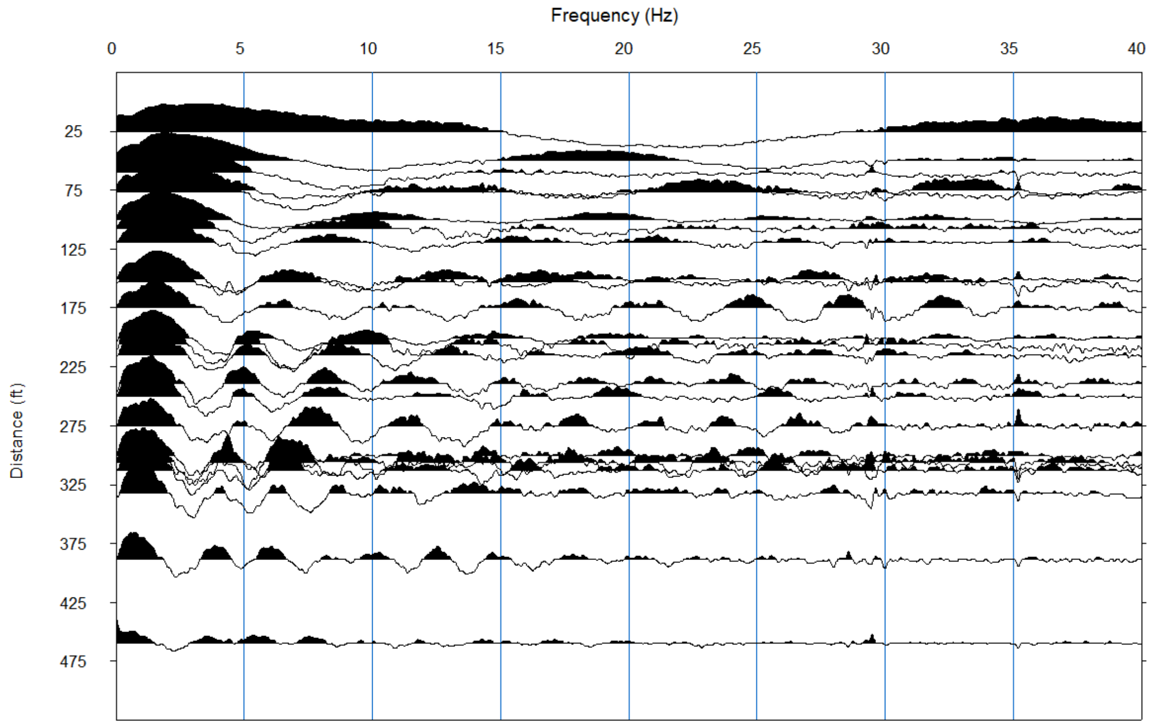


Figure 5.19 Spatial autocorrelation (G-array).

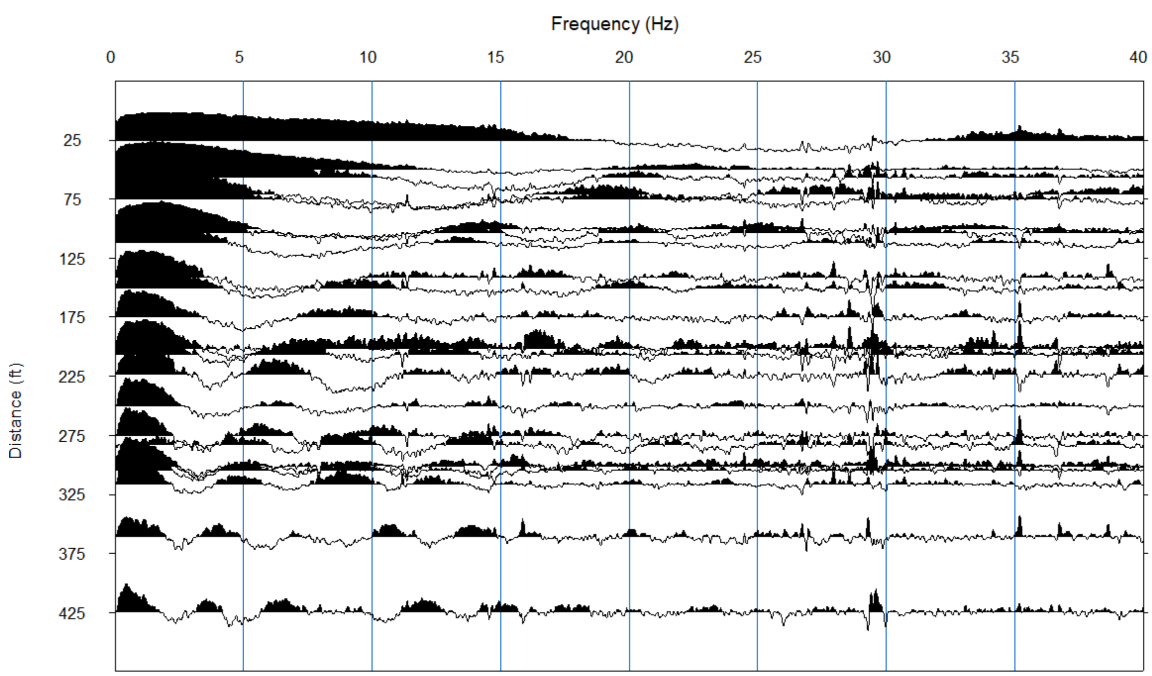


Figure 5.20 Spatial autocorrelation (H-array).

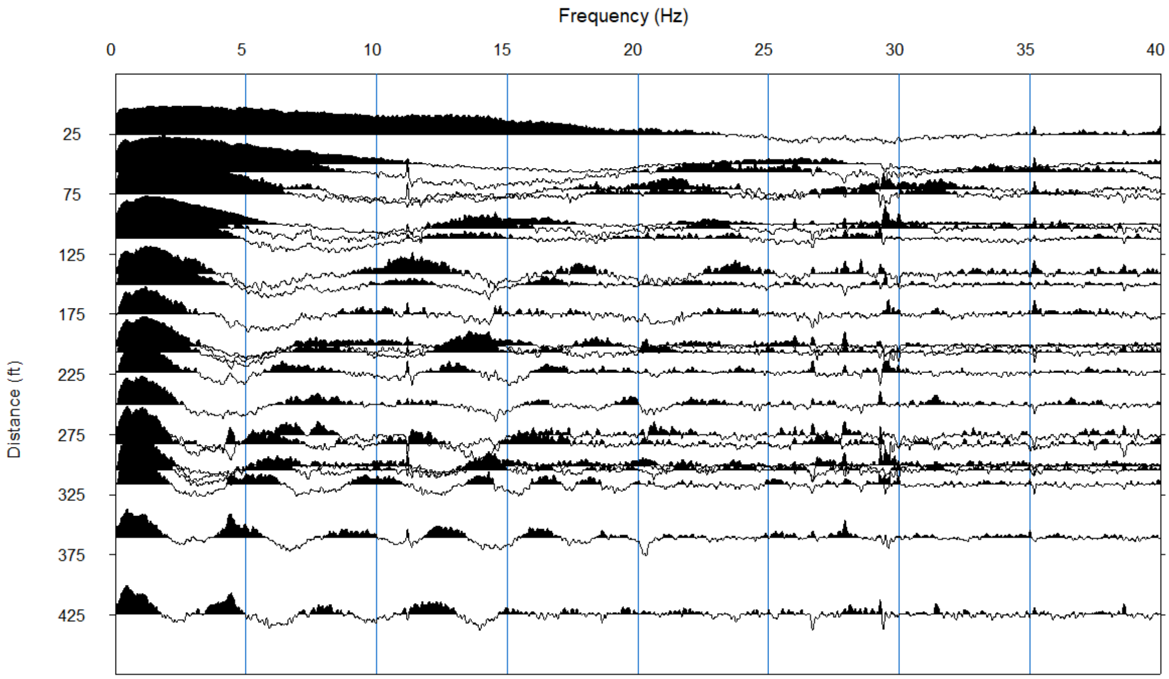


Figure 5.21 Spatial autocorrelation (I-array).

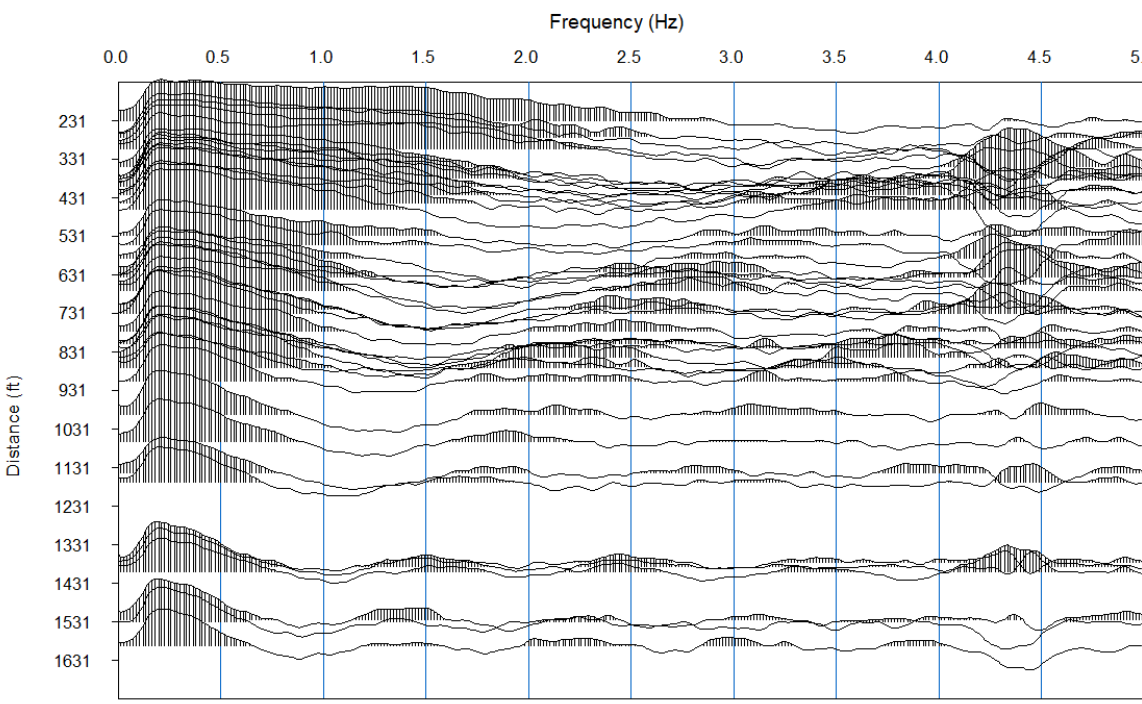


Figure 5.22 Spatial autocorrelation (J-array).

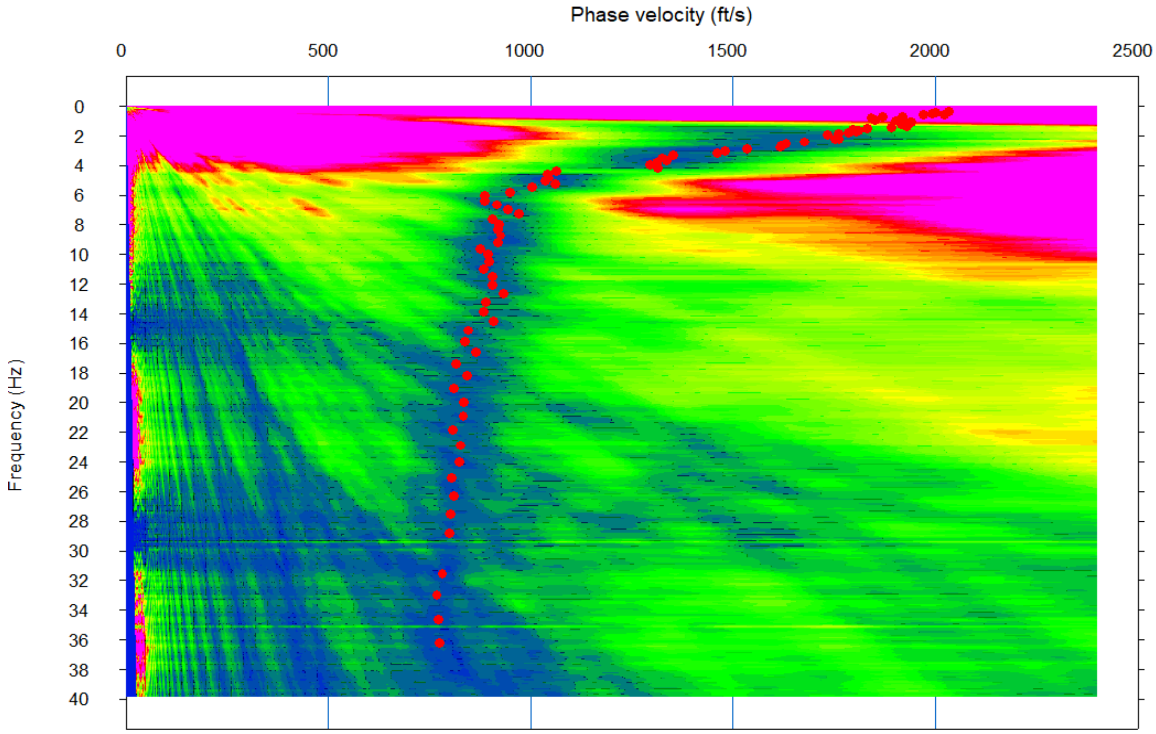


Figure 5.23 Phase velocity image in frequency domain (G-array).

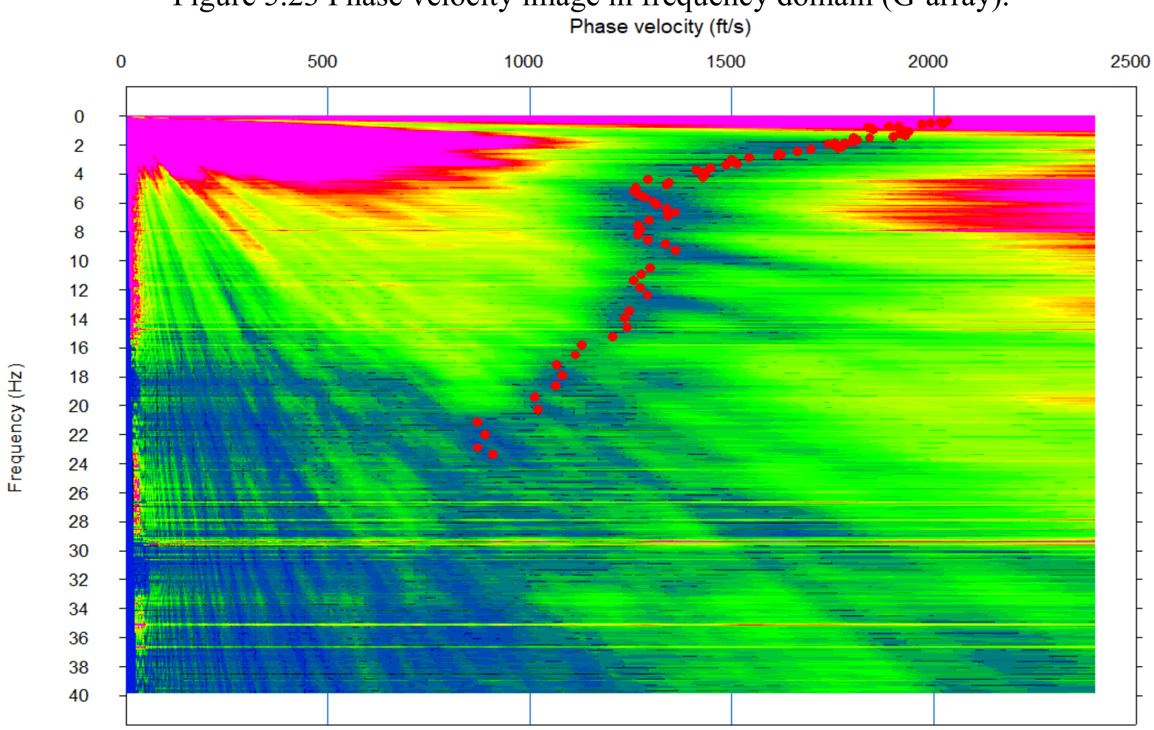


Figure 5.24 Phase velocity image in frequency domain (H-array).

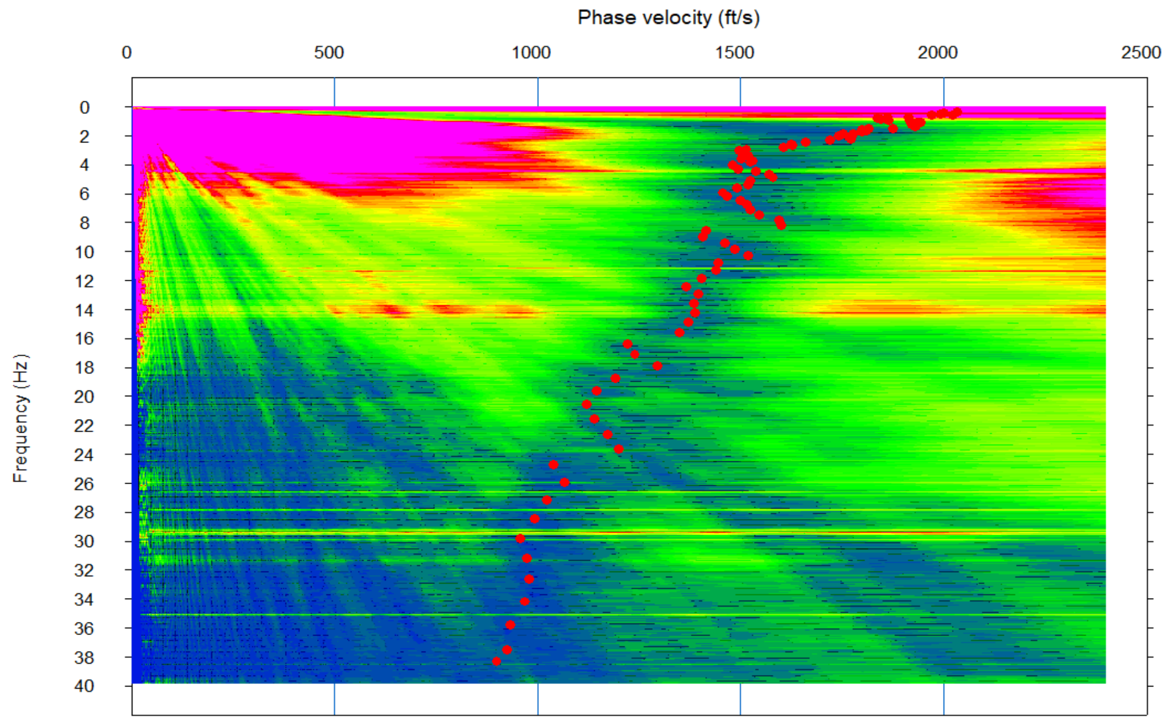


Figure 5.25 Phase velocity image in frequency domain (I-array).

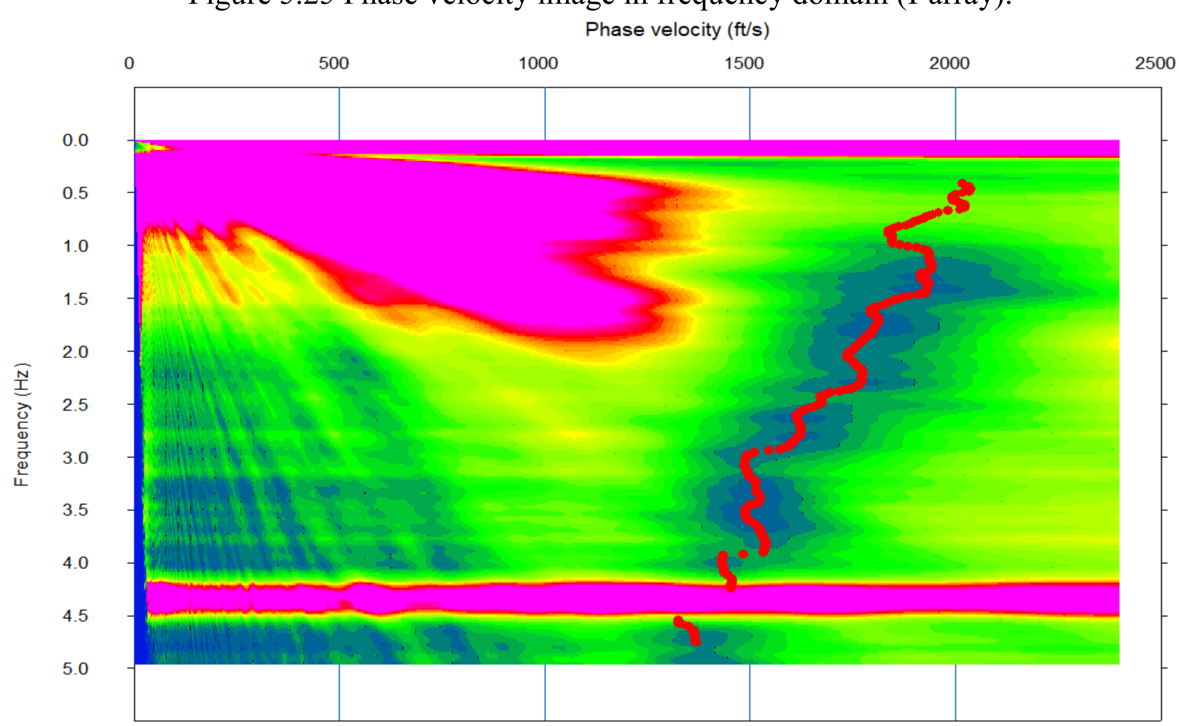


Figure 5.26 Phase velocity image in frequency domain (J-array).

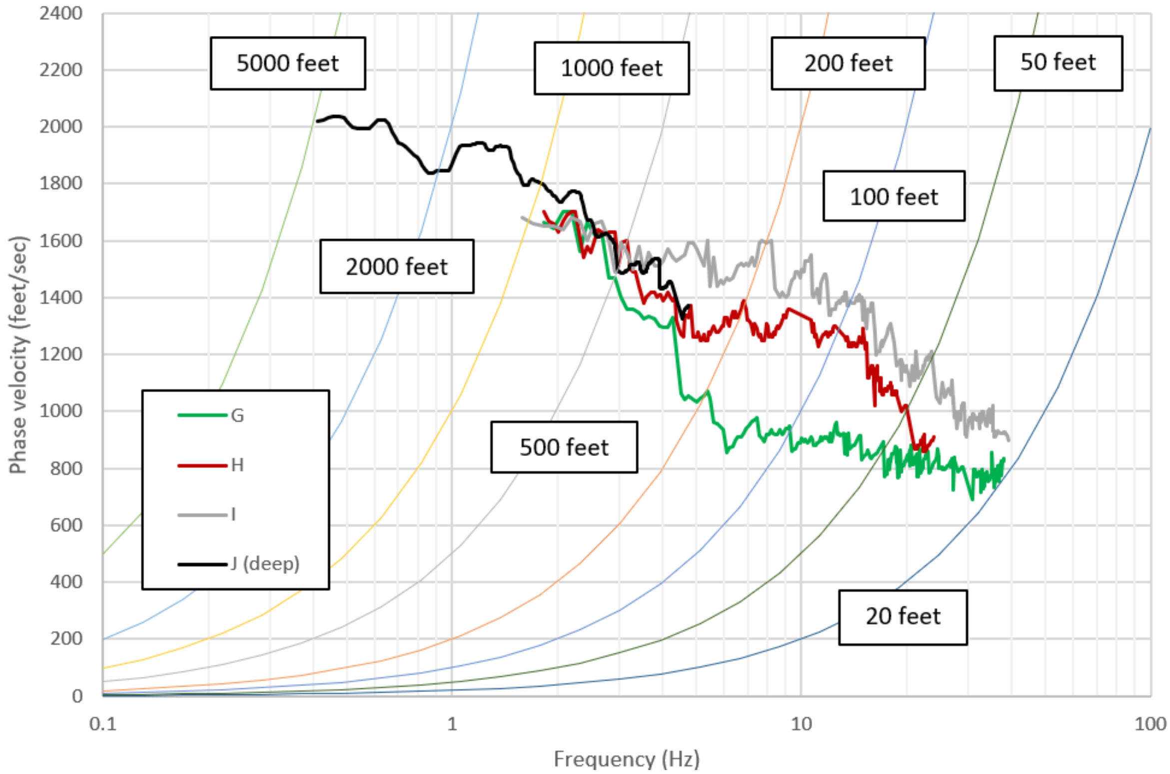


Figure 5.27 Comparison of observed dispersion curves.

5) Analysis results

Figures 5.28 to 5.33 show comparisons of observed and theoretical dispersion curves and inverted velocity models respectively. In the inversion, observed phase velocities (shown as white circles with a red line in Figures 5.28, 5.30 and 5.32) were compared with averaged (effective) phase velocities (shown as yellow circles) taking account of higher modes (shown as color solid lines). We can see that observed and theoretical phase velocities agree well. Green circles on velocity models (Figures 5.29, 5.31 and 5.33) are wavelength divided by three (1/3 wavelength) used for constructing initial velocity modes and they imply approximate depth of investigation. The maximum depth of 1/3 wavelength are more than 400 feet at all arrays and it indicates that the data contains the information down to at least depth of 400 feet.

Figure 5.34 shows a comparison of analyzed S-wave velocity models. In a depth range between 20 and 200 feet, the S-wave velocity at G-array is clearly lower than H- and I-arrays. Figure 5.35 shows the locations of surface wave method lines, microtremor arrays and existed borings. Figure 5.36 shows a comparison of blow counts obtained by the boring and S-wave velocity profiles obtained by surface wave method (MASW) and microtremor array measurements (MAM). In a depth range between 25 and 60 feet, both blow counts and S-wave velocities are larger/higher at the western side of investigation area compared with the eastern side. We can say that the S-wave velocity models obtained by MASW and MAM are generally consistent with the boring logs. The results of surface wave method (MASW) and microtremor array measurements (MAM) can be summarized as follows:

- S-wave velocity (Vs) at the site is basically increasing with depth.
- Vs models basically consist of four layers.
- 1st layer exists shallower than 10 ~ 25 feet. V_S rapidly increases from 600 to 1000 feet/sec in the layer. V_S in the A- and B-lines are lower than the other lines in a depth shallower than 10 feet.
- 2nd layer exists in a depth range between 10 ~ 25 and 60 ~ 70 feet. V_S is almost constant in the layer. V_S in the western side of the investigation area (E- and F-lines, H- and I-arrays) are higher than the eastern side (A-, B-, C-, and D-line, G-array).
- 3rd layer exists in a depth range between 60 ~ 70 feet and 200 feet. Vs increases from 1200 to 1400 feet/sec at G-array and almost constant at H- and I-arrays.

- 4^{th} layer exists at a depth greater than 200 feet. Vs is increasing with depth in the layer.
- V_S is approximately 2000 feet/sec at a depth of 400 feet.
- In a depth range between 25 and 60 feet, both blow counts and S-wave velocities are larger/higher at the western side of investigation area compared with the eastern side.
- Average Vs to a depth of 100 feet (AVS100 feet) is 982.6 feet/sec at the G-array, 1210.0 at the H-array, and 1360.0 feet/sec at the I-array.
- Site class is D at the G-array and C at the H- and I-arrays.

Obtained S-wave velocities, average S-wave velocity to a depth of 100 feet (AVS 100 feet), and the Site Class are summarized in Table 5.2.

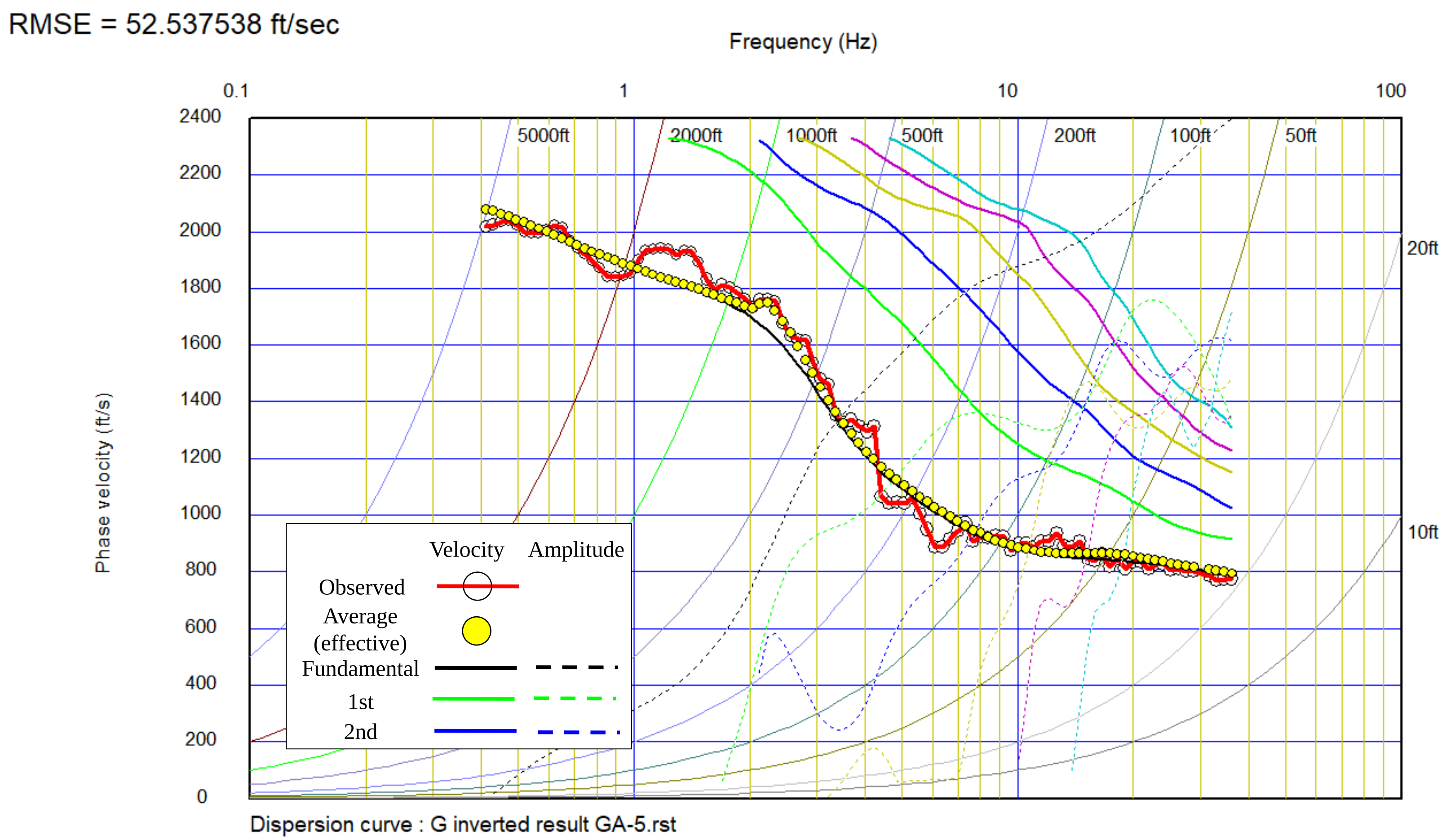


Figure 5.28 Comparison of observed and theoretical dispersion curves (G-array).

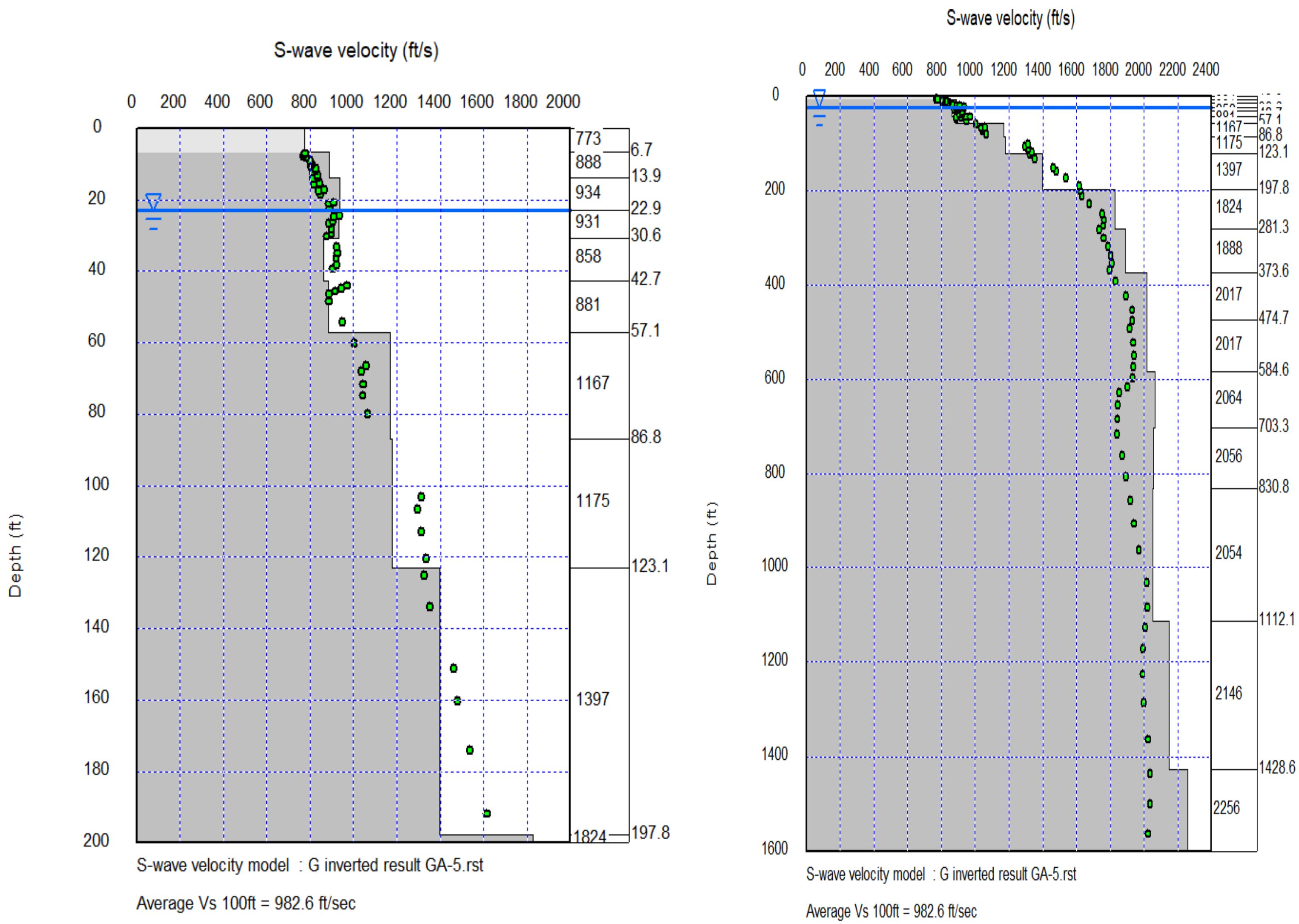


Figure 5.29 Analyzed S-wave velocity model (G-array).

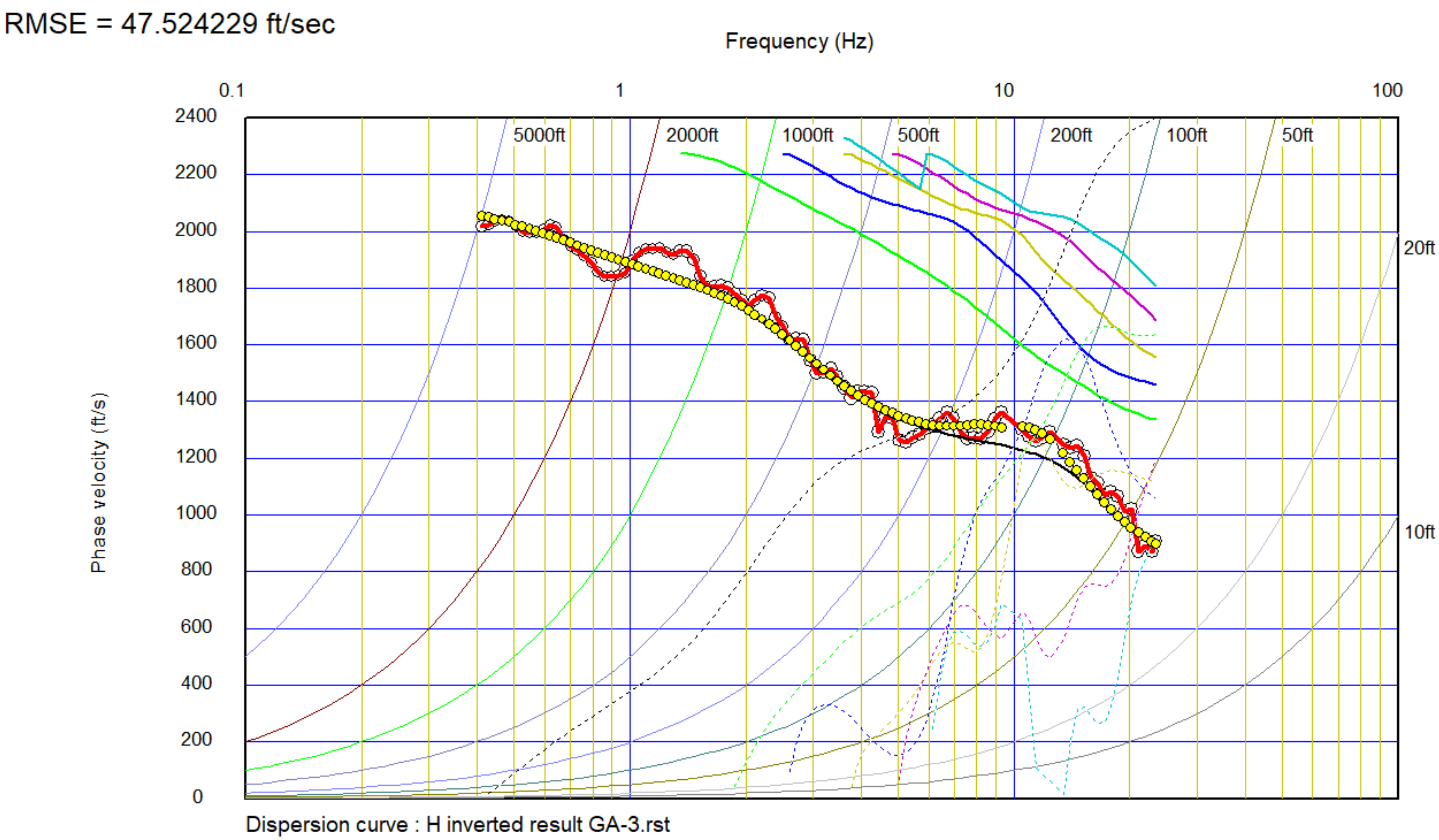


Figure 5.30 Comparison of observed and theoretical dispersion curves (B-array).

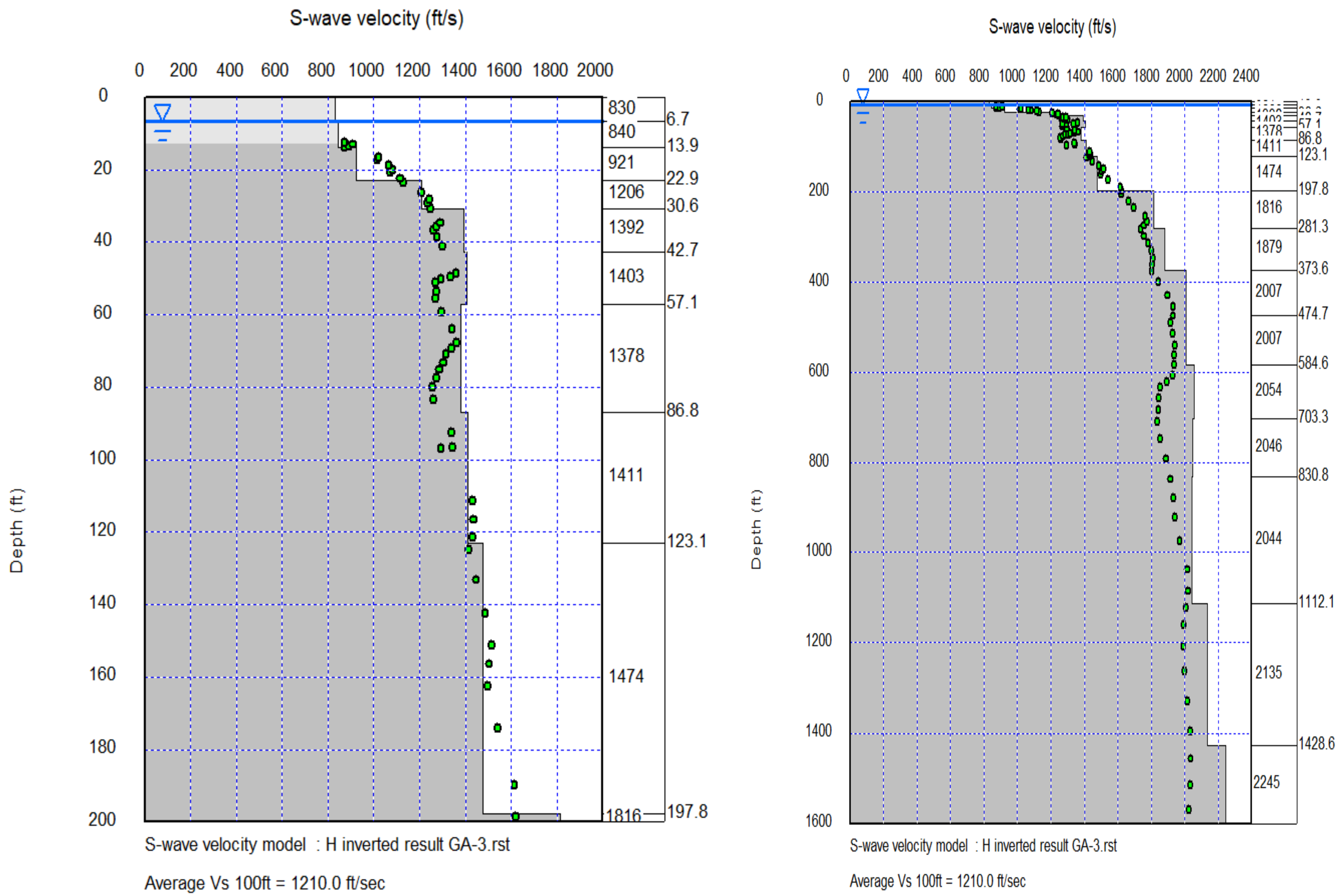


Figure 5.31 Analyzed S-wave velocity model (B-array).

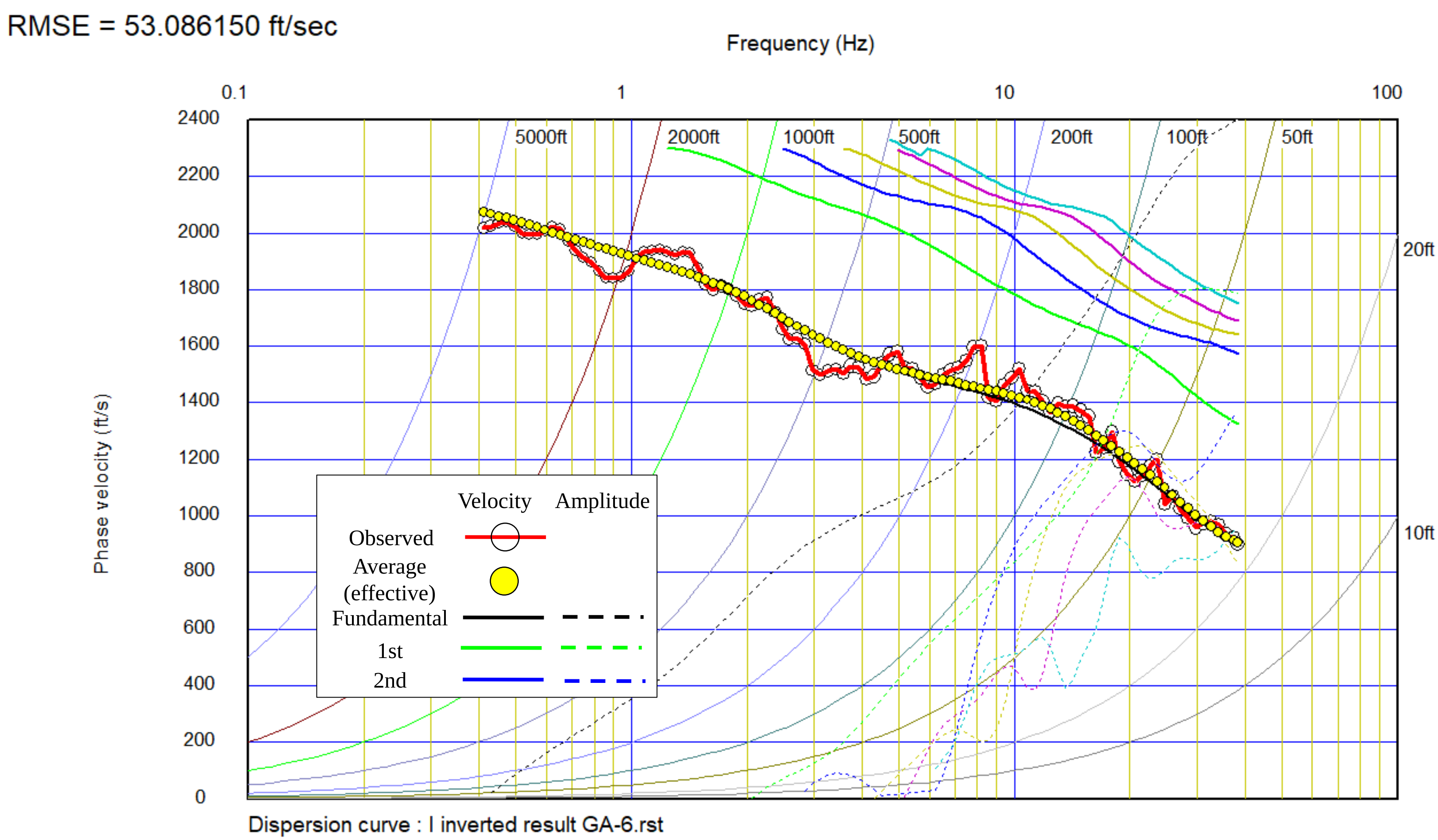


Figure 5.32 Comparison of observed and theoretical dispersion curves (B-array).

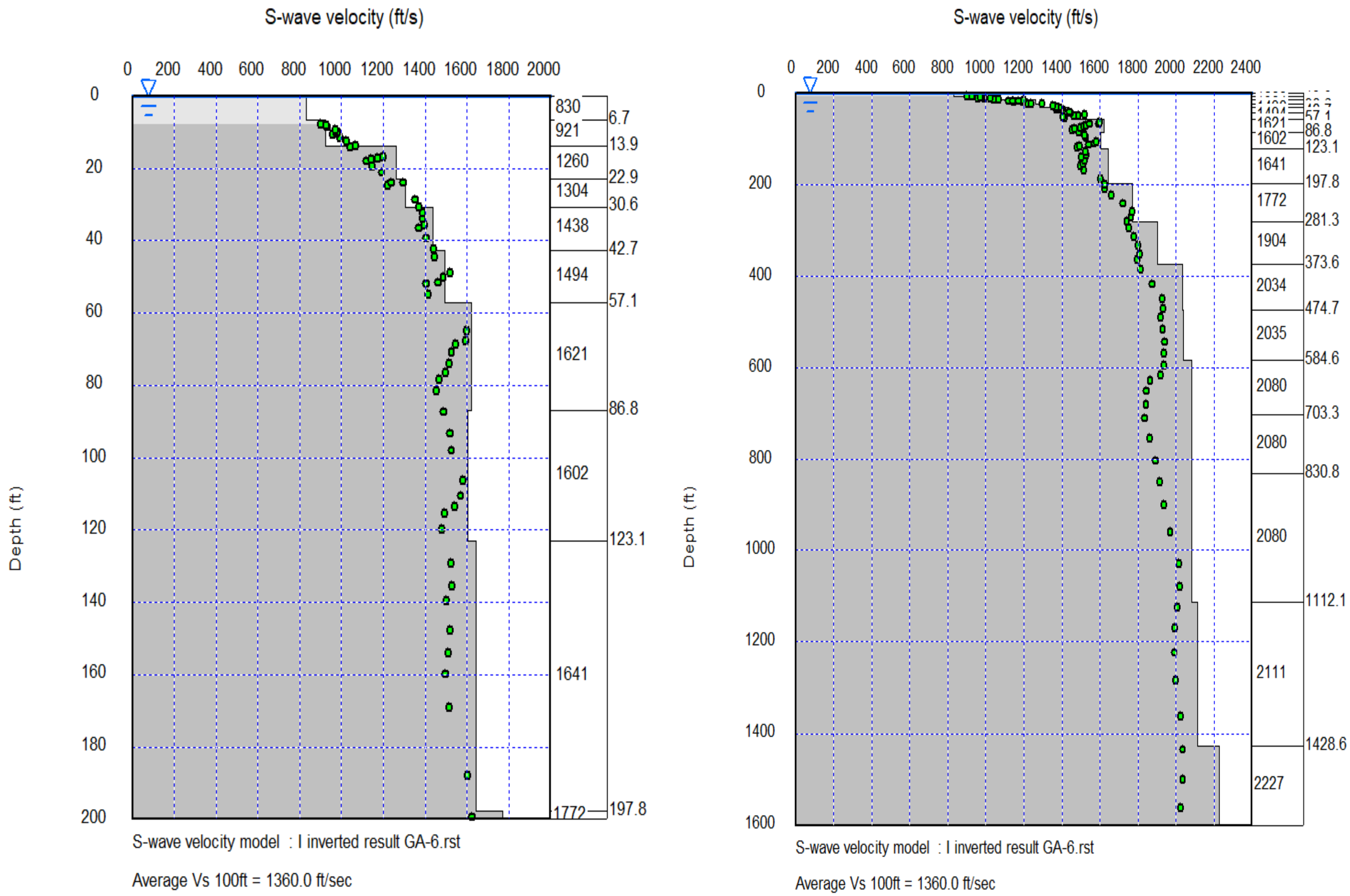


Figure 5.33 Analyzed S-wave velocity model (B-array).

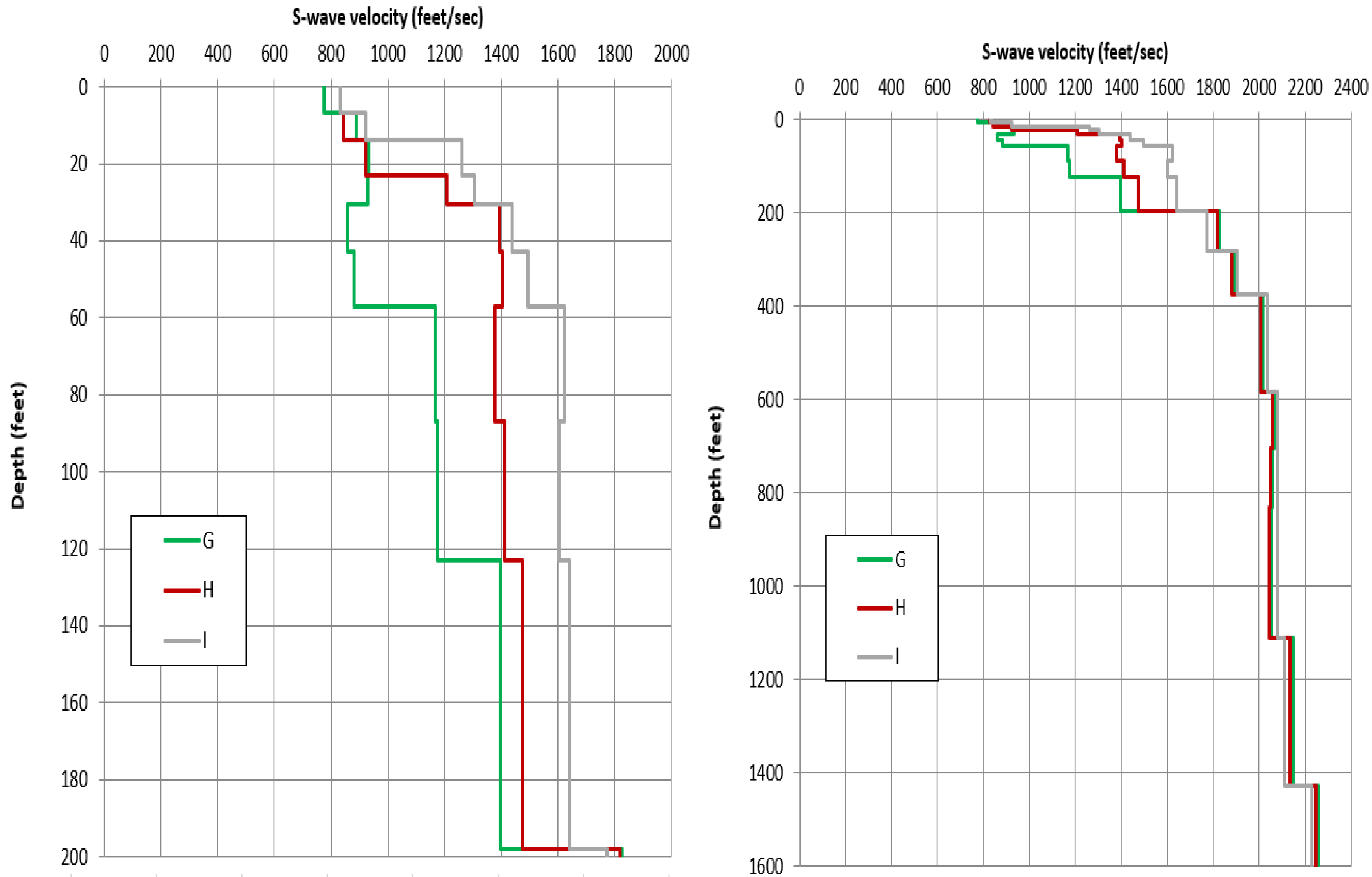


Figure 5.34 Comparison of analyzed velocity models.

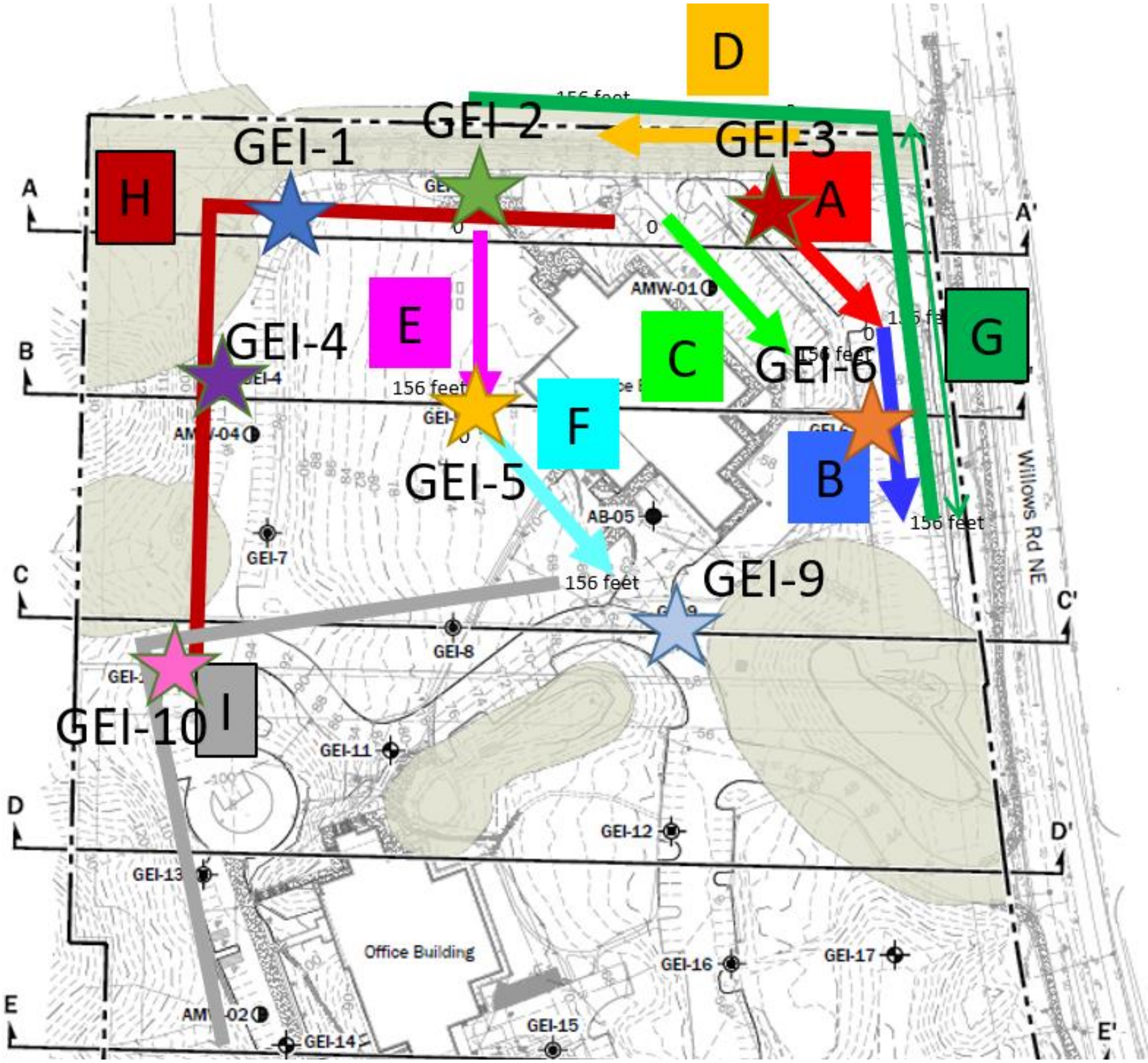


Figure 5.35 Location of borings, surface wave method lines and microtremor arrays.

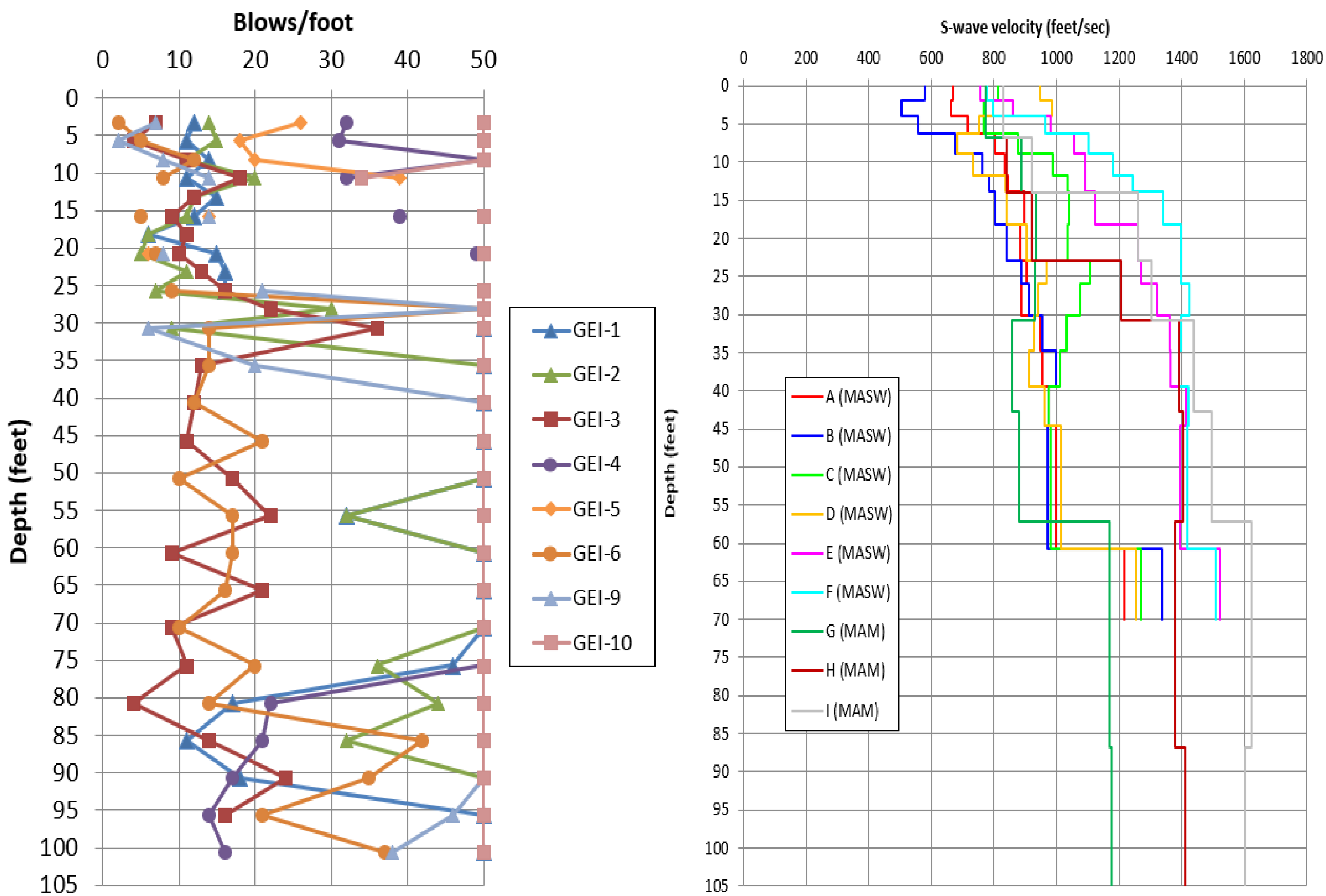


Figure 5.36 Comparison of blow counts obtained by borings (left)
and S-wave velocity profiles obtained by surface wave method microtremor array measurements (right).

Table 5.2 S-wave velocity, average S-wave velocity to a depth of 100 feet (AVS 100 feet), and the Site Class of the sites.

Depth (feet)		S-wave velocity (feet/sec)		
top	bottom	G	H	I
0.0	6.7	773.9	830.1	830.9
6.7	13.9	888.3	841.0	921.9
13.9	22.9	934.9	921.3	1260.4
22.9	30.6	931.1	1206.2	1304.2
30.6	42.7	858.3	1392.0	1438.2
42.7	57.1	881.1	1403.3	1494.7
57.1	86.8	1167.9	1378.9	1622.0
86.8	123.1	1175.6	1411.0	1602.7
123.1	197.8	1397.9	1474.1	1641.7
197.8	281.3	1824.9	1816.1	1772.5
281.3	373.6	1888.3	1879.2	1904.5
373.6	474.7	2017.3	2007.6	2034.1
474.7	584.6	2017.3	2007.6	2035.8
584.6	703.3	2064.6	2054.6	2080.8
703.3	830.8	2056.5	2046.6	2080.8
830.8	1112.1	2054.4	2044.4	2080.8
1112.1	1428.6	2146.2	2135.8	2111.7
1428.6	1942.9	2256.3	2245.3	2227.5
1942.9	-	2331.7	2275.3	2300.3
AVS 100 feet (feet/sec)		982.6	1210.0	1360.0
Site class		D	C	C

5.3 Vibration monitoring

Analysis of vibration monitoring was applied to typical five data summarized in Table 5.3. Each data is 45 seconds in length extracted from continuous measurements. This section summarizes the results of the processing applied to the data shown in the table.

Table 5.3 Typical data shown and processed in the section.

Data	Line	Time	Purpose	Event
A	K	7:22:39 am	Attenuation from a speed bump.	A trailer passed on a speed bump.
B	K	7:23:49 am	Attenuation from a speed bump.	An SUV passed on a speed bump.
C	L	8:03:40 am	Attenuation from the road during a noisy period. Data can be considered as the largest vibration from the road.	A large track passed through a road.
D	L	0:40:42 am	Attenuation from the road during a quiet period. Data can be considered as background noise without any artificial noises.	No traffic.
E	M	6:32:17 pm	Attenuation from a road during a noisy period at another site. Data can be considered as the largest vibration from a road.	A large bus passed through a road.

1) Raw data

Figures 5.37 to 5.41 show raw waveform data shown in Table 5.3. Four seismographs were used in the measurements and each seismograph has three components, so that 12 traces are shown in each data. Maximum amplitude (acceleration) in each trace is shown at the righthand side of the figures. Four seismographs were deployed on lines K, L and M. Traces are shown as getting away from the sources, a speed bump or a road, so that the amplitude of waveforms generally decreases from top to bottom of the figures except with data D. The amplitude of data D is almost constant throughout the four seismographs since the data was recorded at midnight and only contains background noise without traffic noises.

2) Octave analysis

Figures 5.42 to 5.49 show the results of an octave analysis. The amplitude is shown as acceleration. Figure 5.45, 5.47 and 5.49 show the results as velocity and displacement compared with acceleration. Figures 5.45 and 5.49 show that the maximum traffic noise during busy traffic conditions. At a distance of 100 feet from the road, the amplitude is approximately 0.1 ~ 1.0 gal (cm/sec^2) in acceleration, 0.004 ~ 0.1 kine (cm/sec) in velocity, and 0.0007 to 0.004 mm in displacement in a frequency range between 5 and 20 Hz.

Figure 5.47 shows that background noise at quiet time. Amplitude is approximately 0.06 gal (cm/sec^2) in acceleration, 0.0003 kine (cm/sec) in velocity, and 0.00003 mm in displacement in a frequency range between 5 and 20 Hz.

3) Attenuation with distance

Figure 5.50 shows the attenuation of vibration level from a speed bump with distance along the K-line. Raw data are shown in Figures 5.37 and 5.38 and the results of the octave analysis are shown in Figures 5.42 and 5.43. We can see that the vibrations from a speed bump quickly attenuate and they are the same or smaller than background noise beyond 50 feet from the speed bump.

4) Difference associated with traffic condition

Figure 5.51 shows the difference of vibration level associated with traffic condition at the L-line (north entrance). Noise during the noisy period (morning) is approximately 5 ~ 10 times larger than the quiet period (night). During the noisy period (morning), the vibrations from traffic on the road quickly attenuate and they are the same or smaller than background noise beyond 100 feet from the road. In quiet period (night), the vibration level does not change with distance from the road.

5) Difference among sites

Figure 5.52 compares the vibration level between two different sites; L-line (north entrance) and M-line (south entrance). It is clear that the amplitude attenuates with distance from the road. At the distances of 0 and 100 feet, the amplitude level at the south entrance (M-line) is several times larger than the north entrance (L-line).

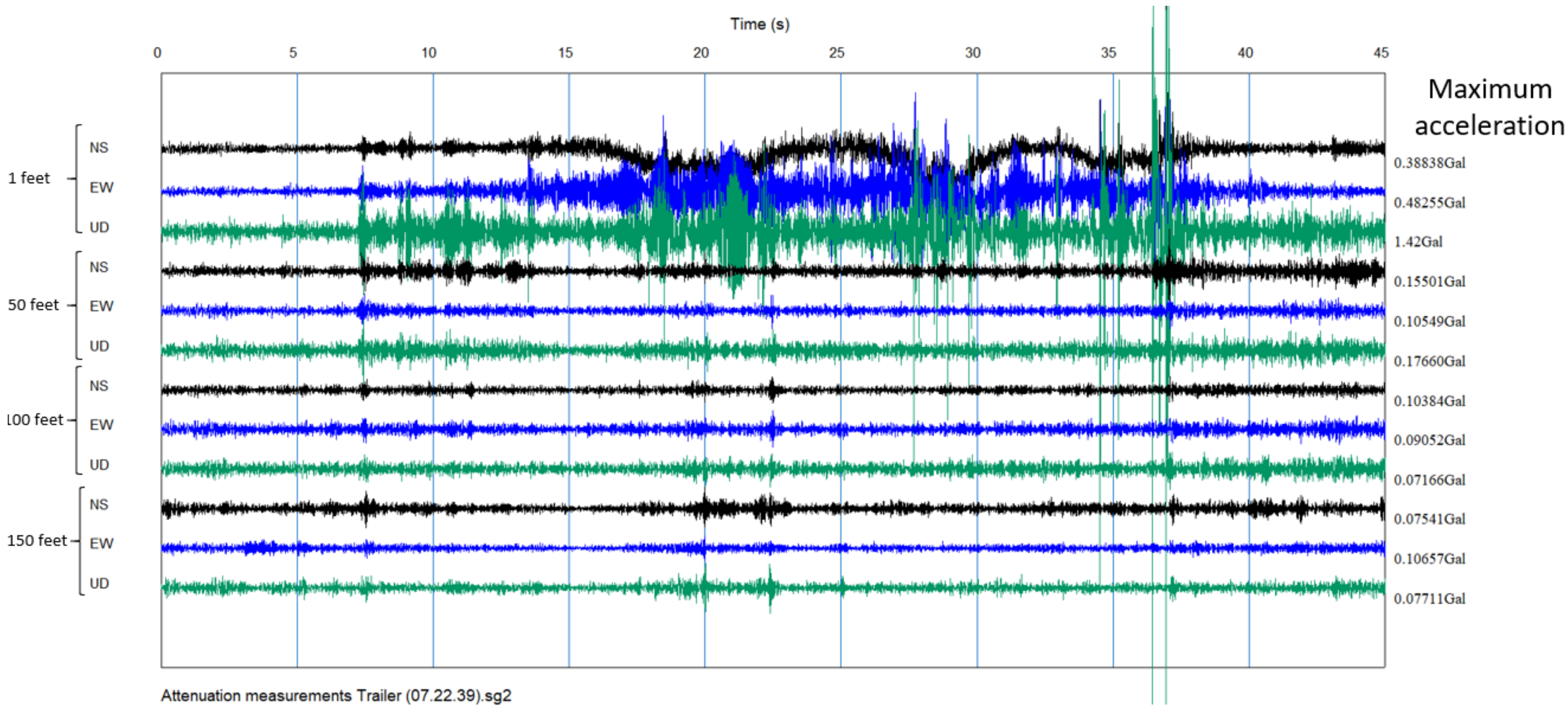


Figure 5.37 Raw wave form data A (K-line : vibration from a trailer on a speed bump (07:22:39)).

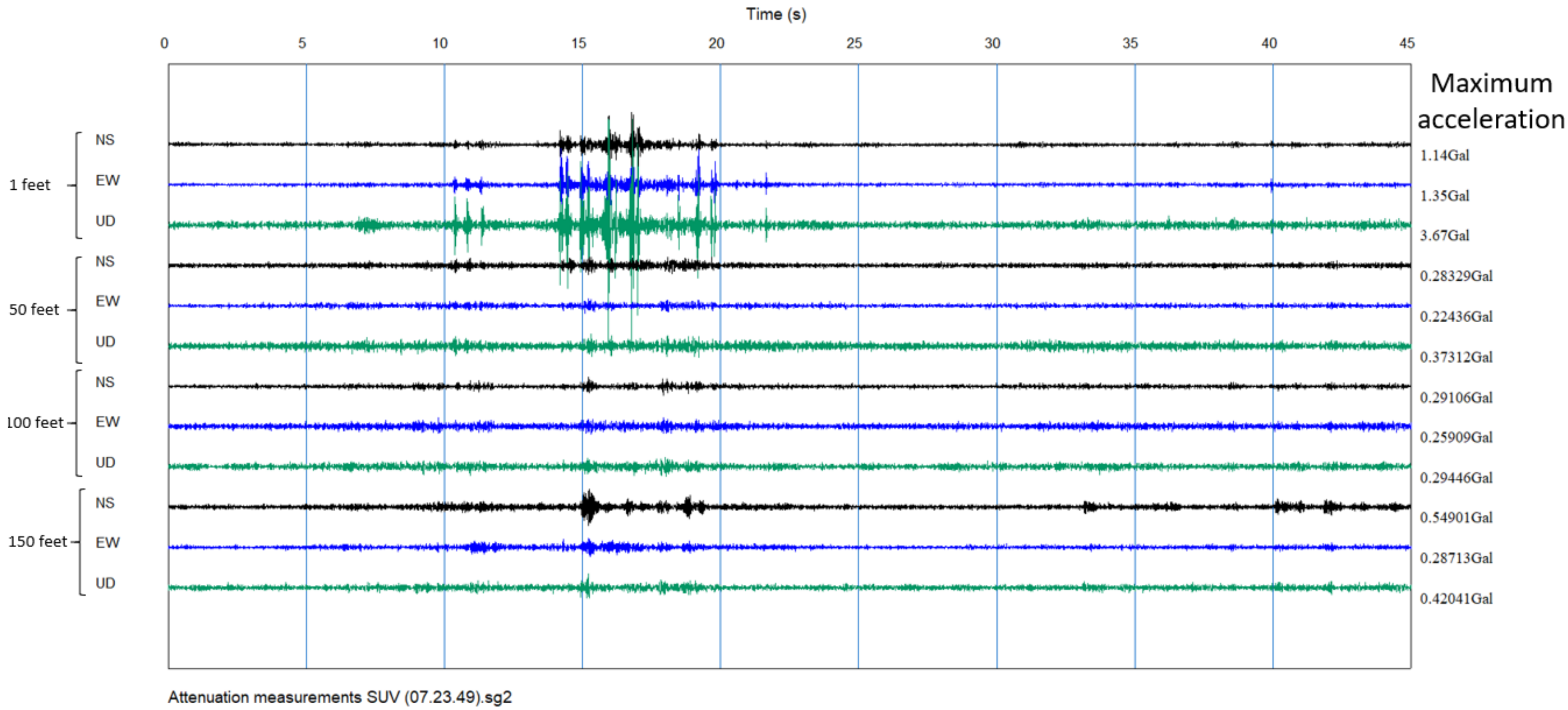


Figure 5.38 Raw wave form data B (K-line: Vibration from an SUV on a speed bump (07:23:49)).

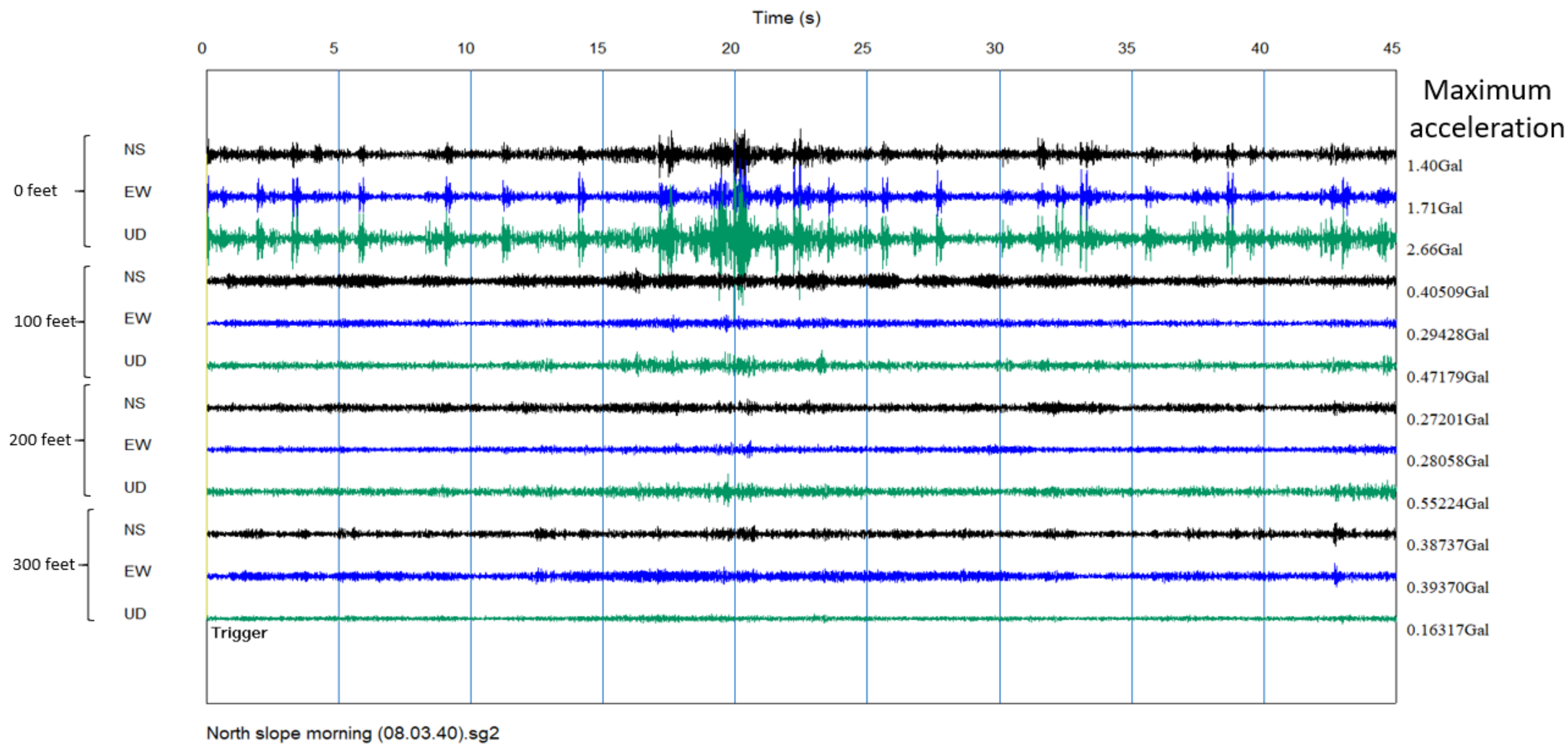


Figure 5.39 Raw wave form data C (L-line: busy traffic in morning (08:03:40)).

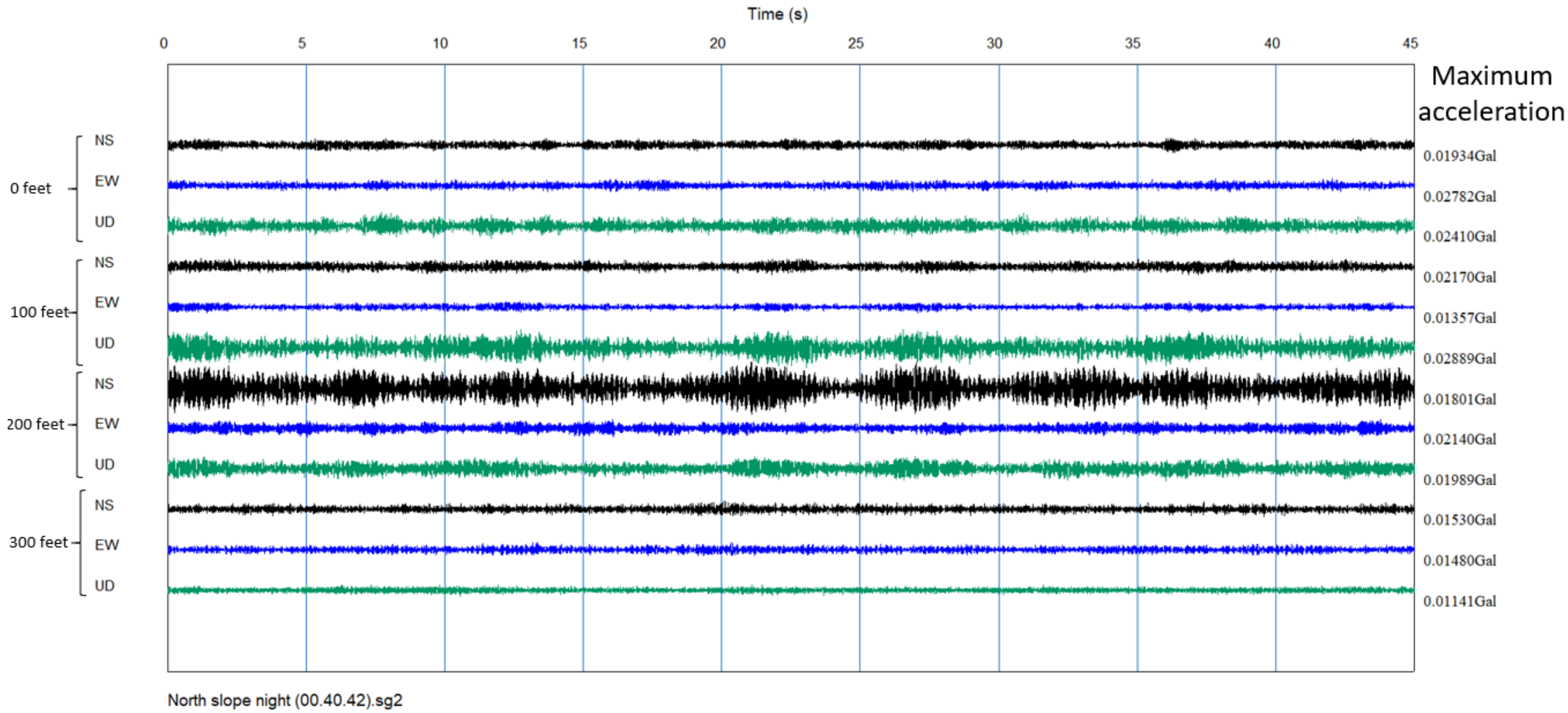


Figure 5.40 Raw wave form data D (L-line: quiet period during late night (00:40:42)).

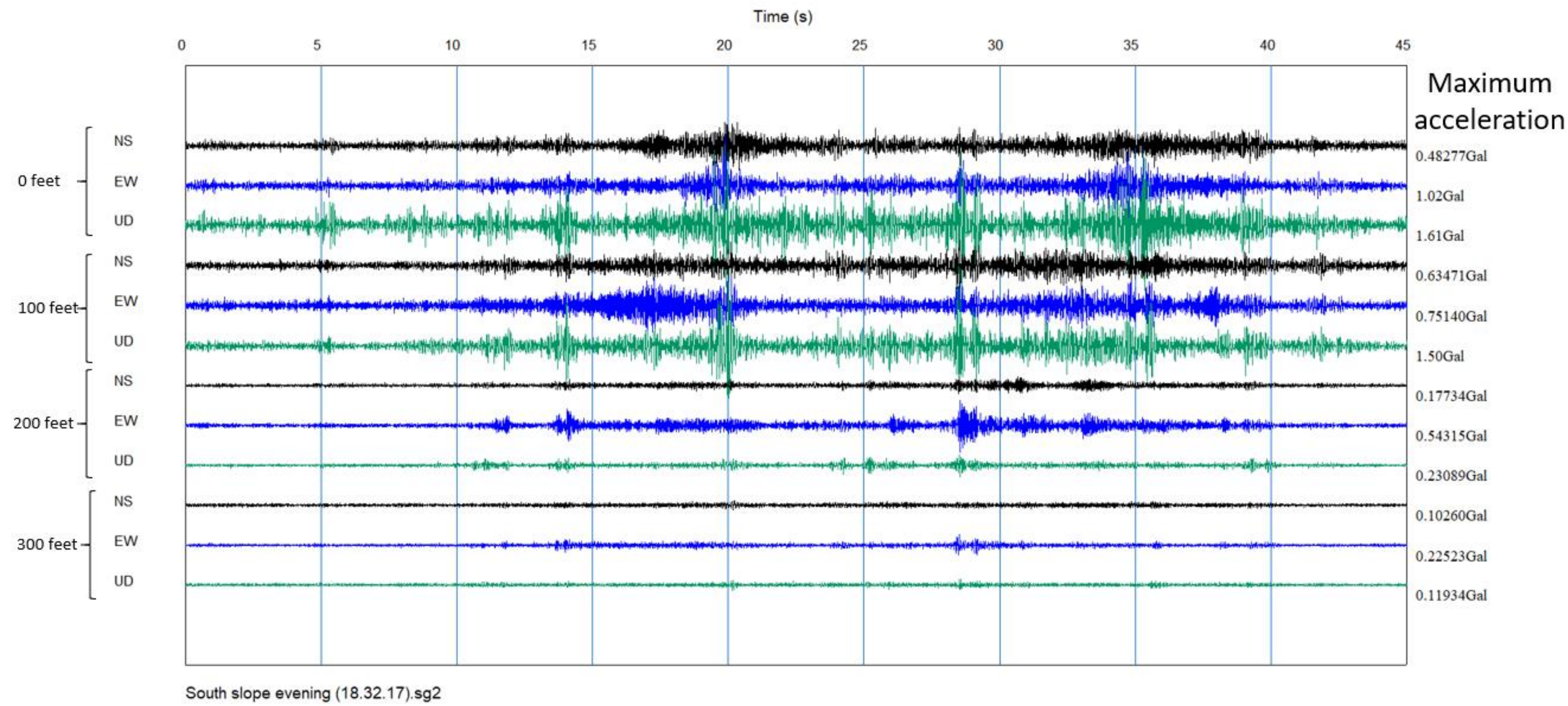


Figure 5.41 Raw wave form data E (M-line: busy traffic in the evening (18:32:17)).

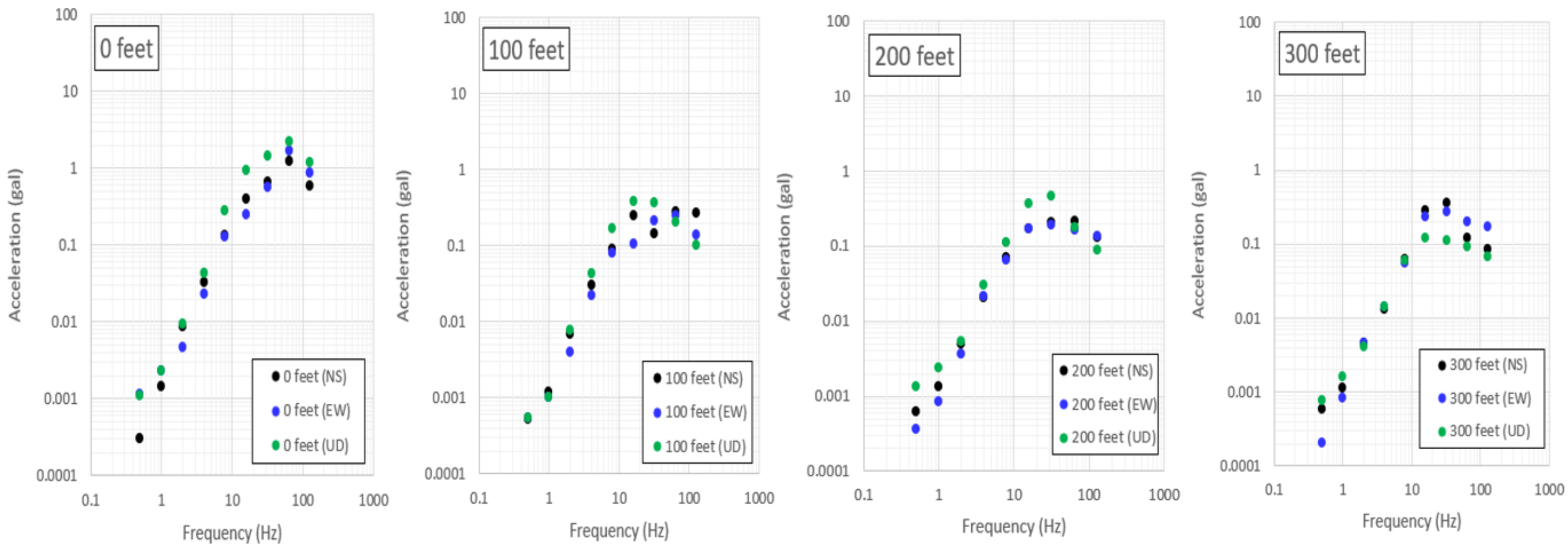


Figure 5.42 Result of octave analysis, A (K-line: vibration from a trailer on a speed bump (07:22:39)).

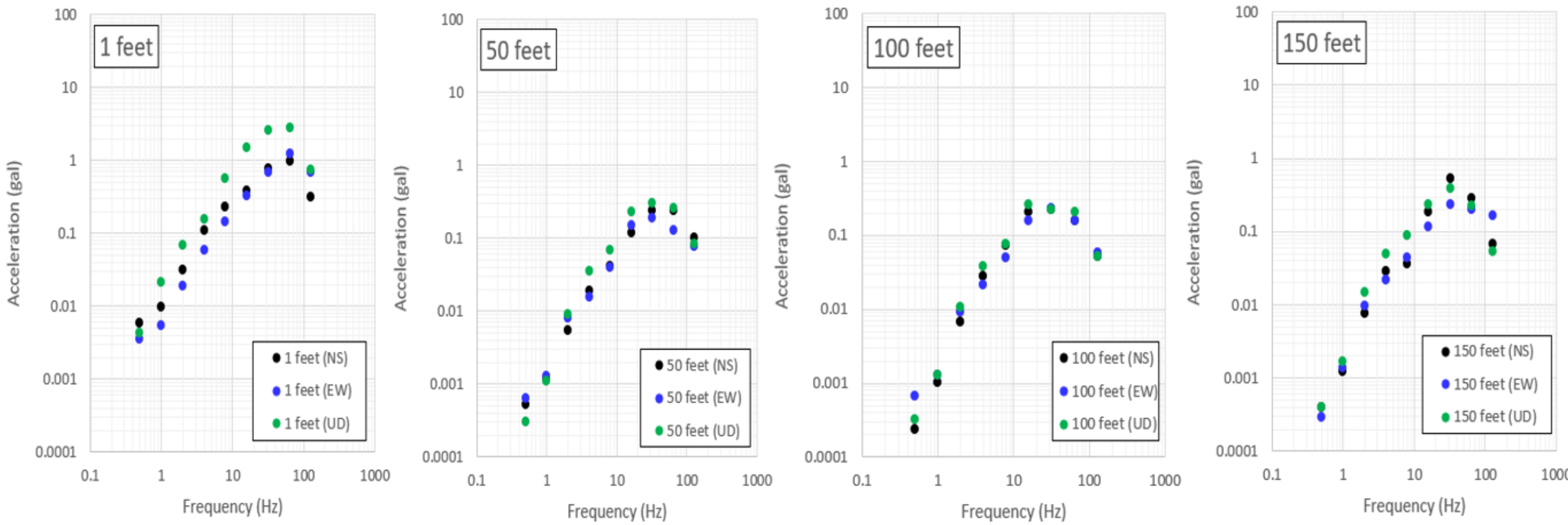


Figure 5.43 Result of octave analysis, B (K-line: Vibration from an SUV on a speed bump (07:23:49)).

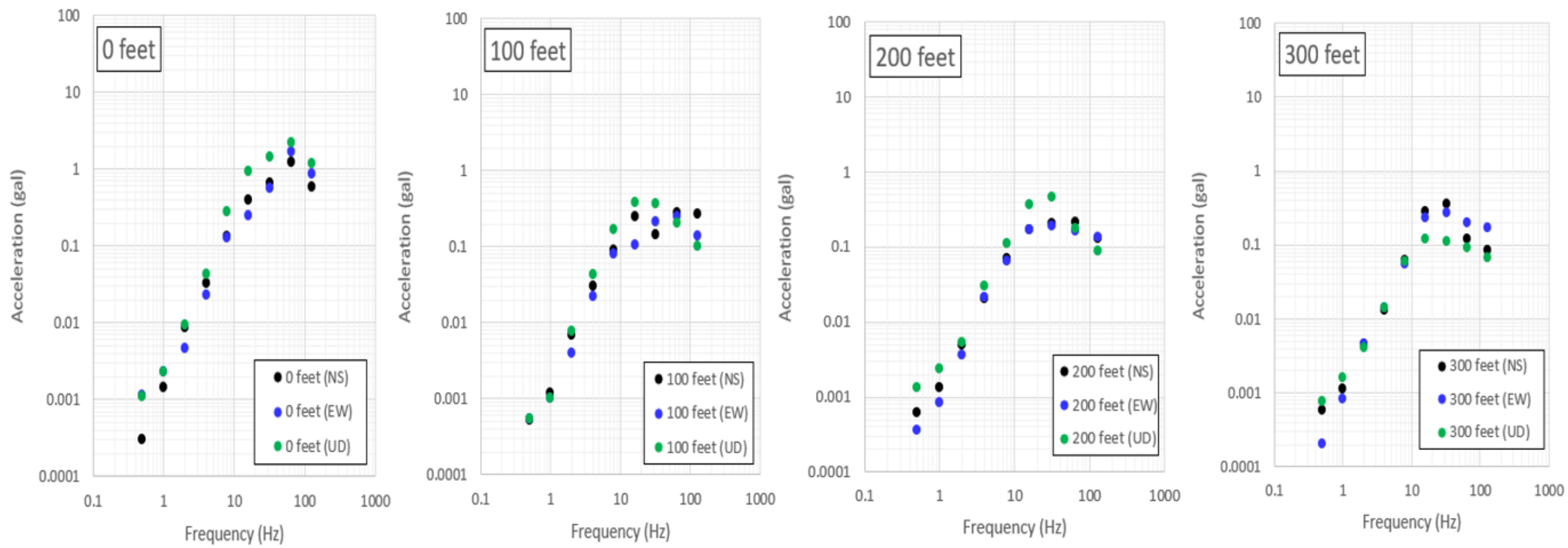


Figure 5.44 Result of octave analysis, C (L-line: busy traffic in the morning (08:03:40)).

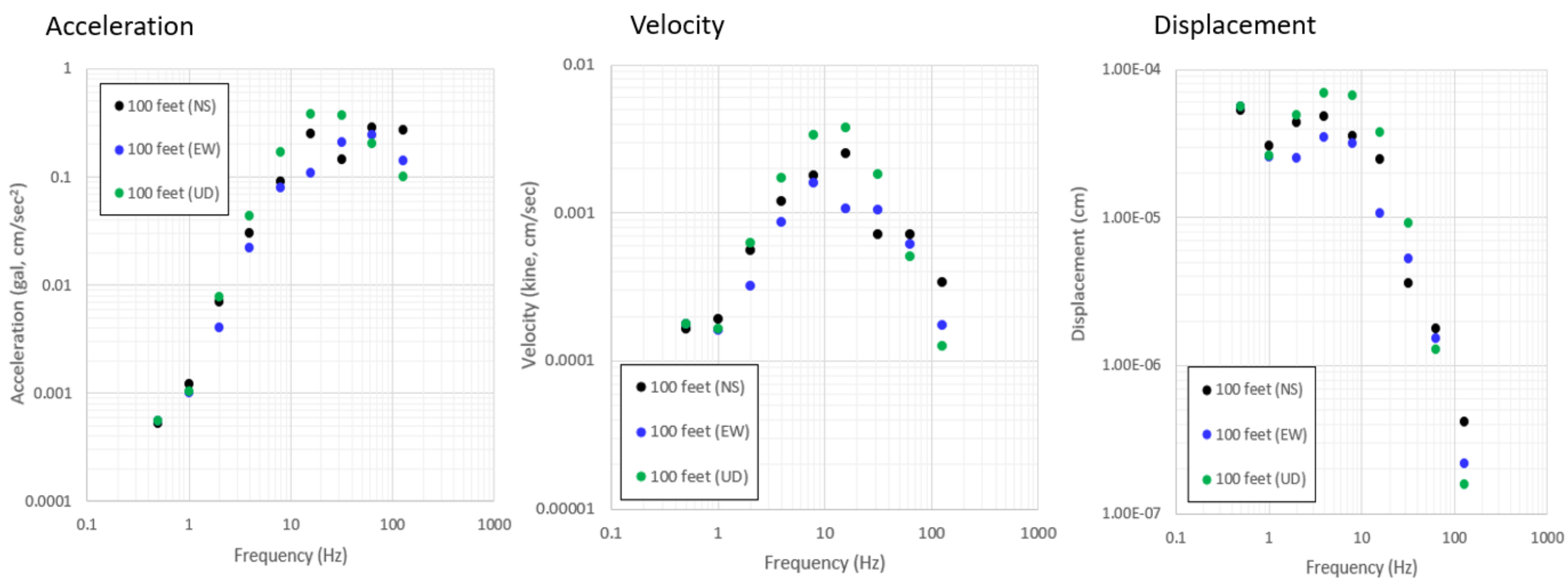


Figure 5.45 Comparison of acceleration, velocity and displacement, C (L-line: busy traffic in the morning (08:03:40)).

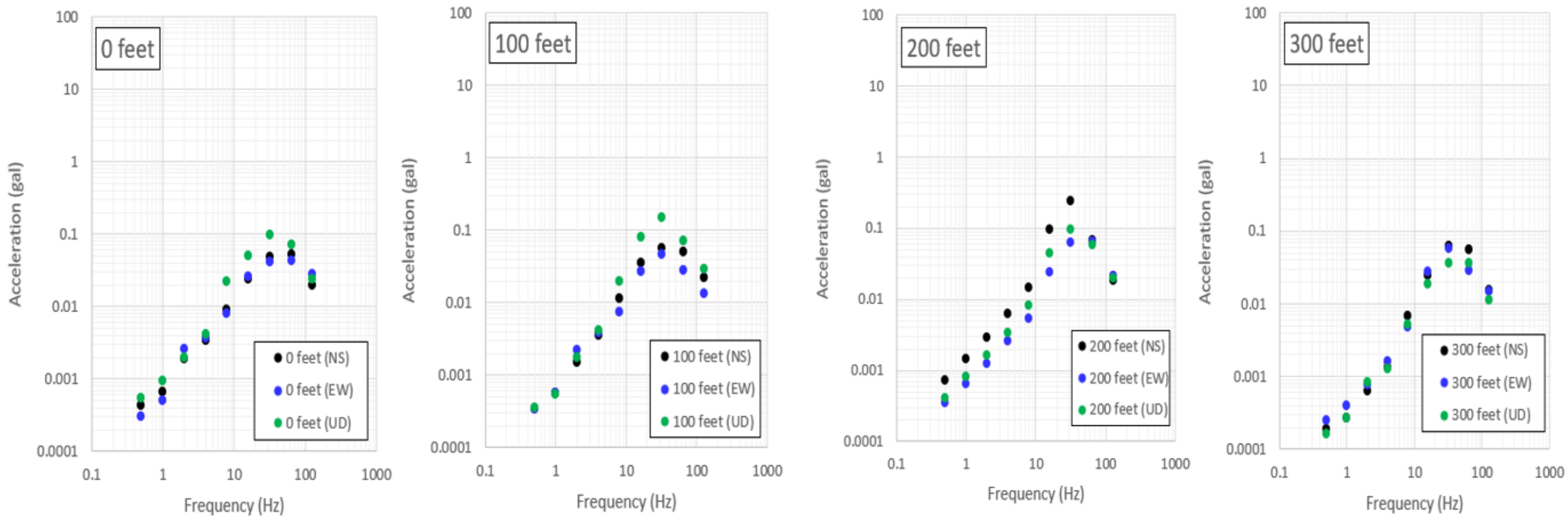


Figure 5.46 Result of octave analysis D (L-line: quiet period during late night (00:40:42)).

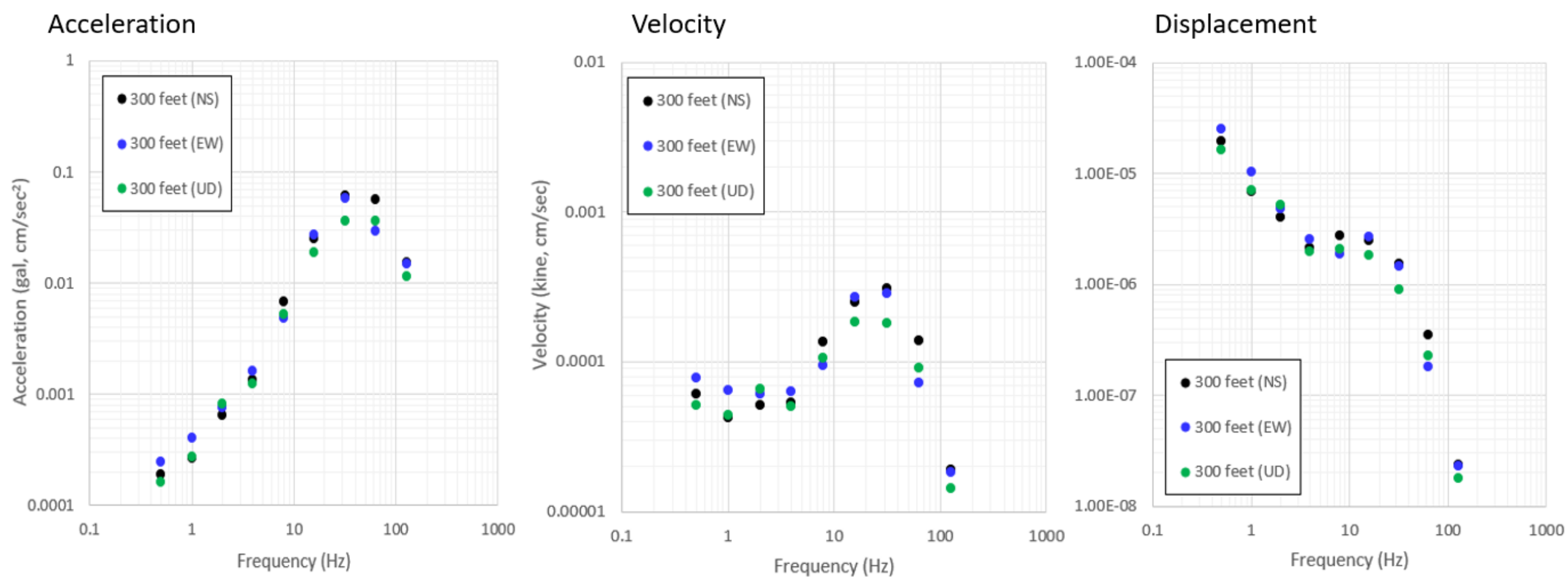


Figure 5.47 Comparison of acceleration, velocity and displacement, D (L-line: quiet period during late night (00:40:42)).

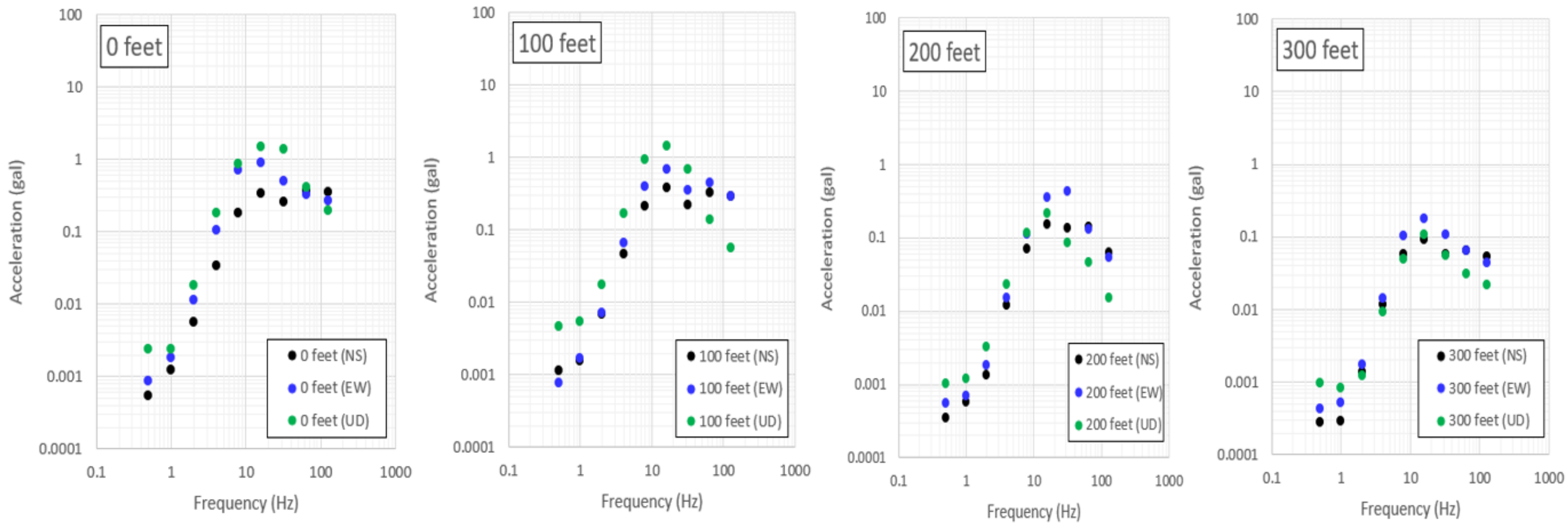


Figure 5.48 Result of octave analysis E (M-line: busy traffic in the evening (18:32:17)).

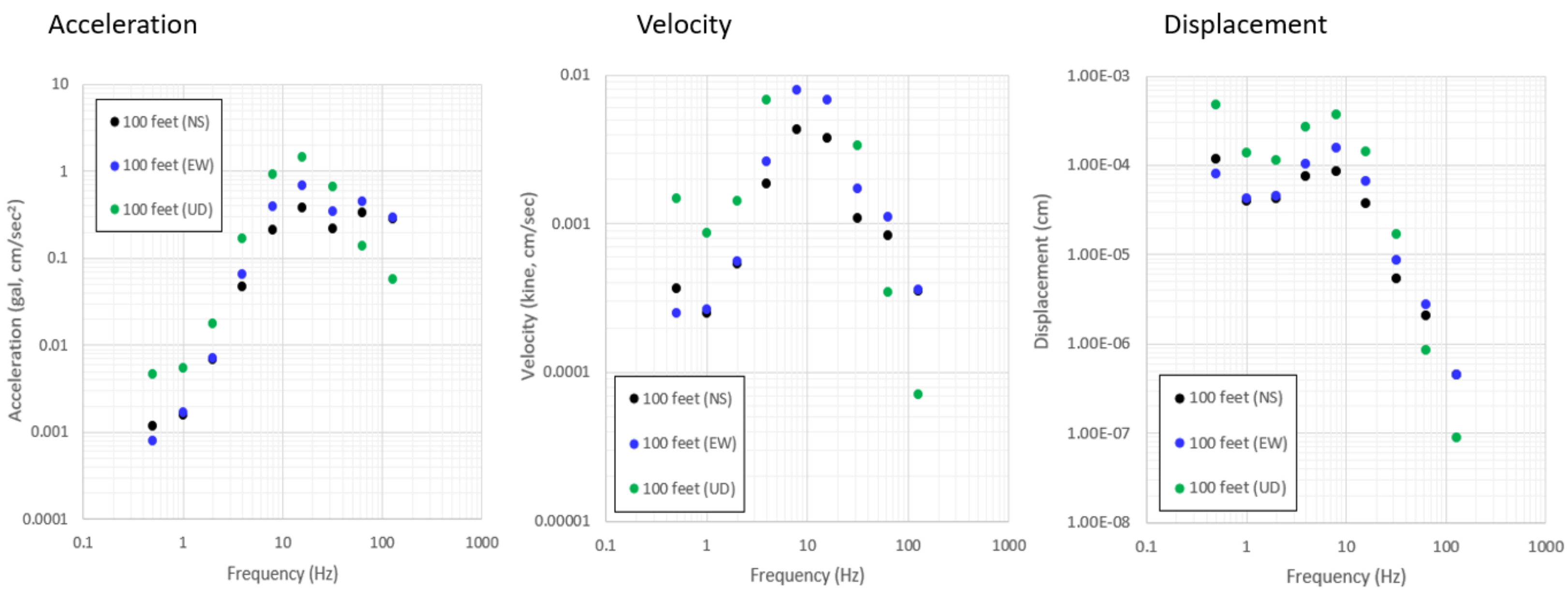


Figure 5.49 Comparison of acceleration, velocity and displacement, E (M-line: busy traffic in the evening (18:32:17)).

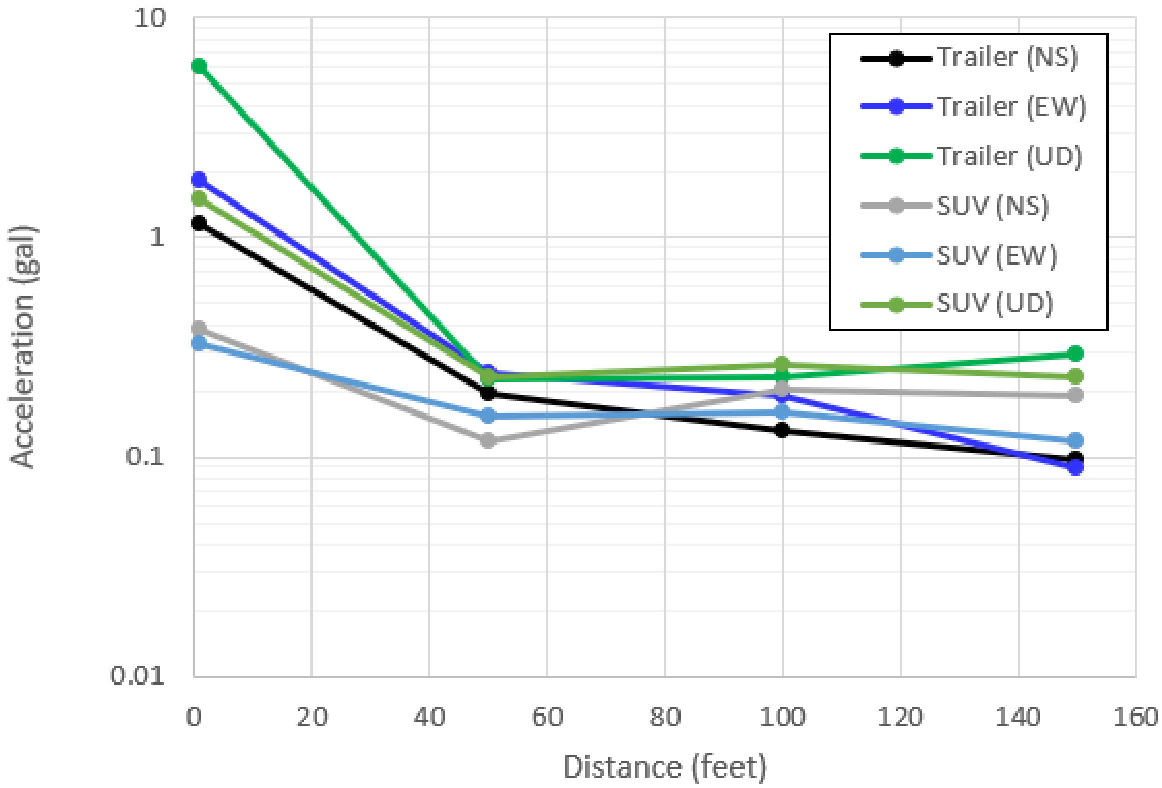


Figure 5.50 Attenuation of vibration level from a speed bump with distance along L-line (16 Hz : 11.3 ~ 22.6 Hz).

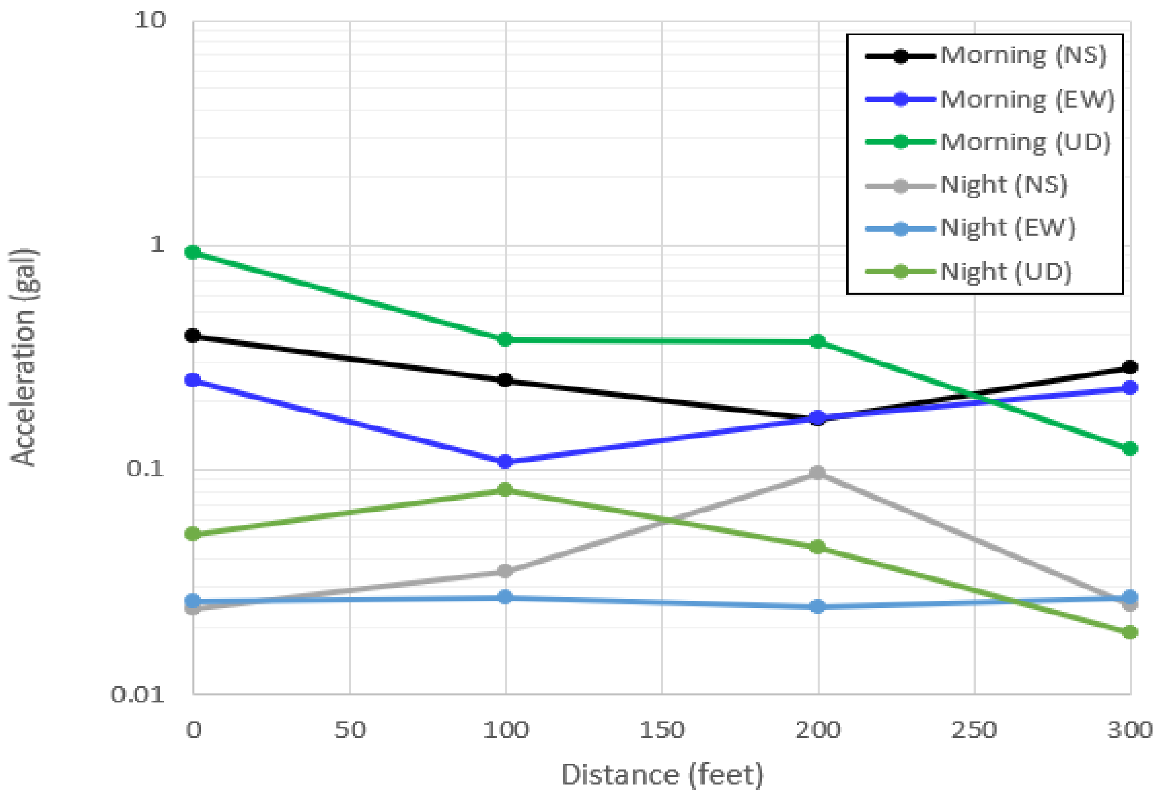


Figure 5.51 Difference of vibration level associated with traffic condition
(16 Hz: 11.3 ~ 22.6 Hz).

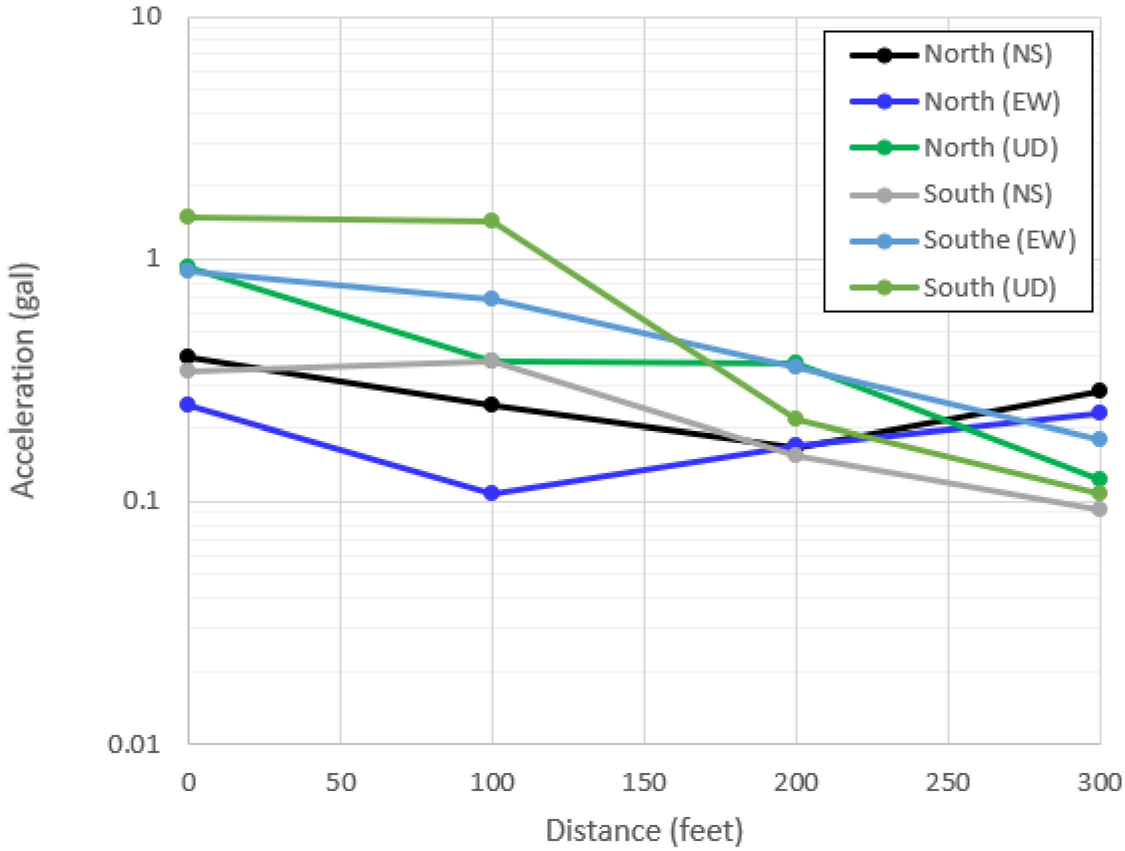


Figure 5.52 Difference of vibration level among sites (16 Hz: 11.3 ~ 22.6 Hz).

6. Conclusions

- There is no significant horizontal velocity change at shallow depth (< 10 feet) in each surface wave method (MASW) line.
- S-wave velocity is generally slower at eastern side of the site in a depth range between 10 ~ 25 feet and 60 ~ 70 feet.
- S-wave velocity is slower at array G (eastern side) in a depth range between 30 and 200 feet.
- S-wave velocity is approximately 2000 feet/sec at a depth of 400 feet.
- AVS 100 feet ranges between 982 and 1360 feet/sec.
- Site class is D at the eastern side (G-array) and C at the western side (H- and I-arrays).
- Vibrations from a speed bump quickly attenuate and they are the same or smaller than background noise beyond 50 feet from the speed bump.
- Vibrations from traffic on a road quickly attenuate and they are the same or smaller than background noise beyond 100 feet from the road.
- Maximum traffic noise at a distance of 100 feet from a road is approximately 0.1 ~ 1.0 gal (cm/sec2) in acceleration, 0.004 ~ 0.1 kine (cm/sec) in velocity, and 0.0007 to 0.004 mm in displacement in a frequency range between 5 and 20 Hz.
- Background noise at quiet nighttime is approximately 0.06 gal (cm/sec^2) in acceleration, 0.0003 kine (cm/sec) in velocity, and 0.00003 mm in displacement in a frequency range between 5 and 20 Hz.

7. References

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APPENDIX E
Site-Specific Response Analysis

APPENDIX E
SITE-SPECIFIC RESPONSE ANALYSIS

A site-specific risk-targeted maximum-considered earthquake (MCE_R) response spectrum was developed per American Society of Civil Engineers (ASCE) 7-16 Chapter 21.

Ground Motion Analysis Procedure

1. Compute basin amplification factors (AFs) to account for ground motion amplification associated with location of the site within the Seattle basin.
2. Complete a site-specific probabilistic seismic hazard analysis (PSHA) with basin-adjusted ground motion models (GMMs) to compute a basin-adjusted firm ground uniform hazard response spectrum (UHS) for the maximum-considered earthquake (MCE) (i.e., 2 percent probability of exceedance in 50 years, 2,475-year return period). Firm ground conditions are defined as Vs30=701 meters per second (m/sec), where Vs30 is the time-averaged shear wave velocity of the upper 100 feet (30 meters) of soil and correspond to the shear wave velocity at the base of the measured profiles.
3. Complete seismic hazard deaggregation for the MCE at structural periods of interest and select a suite of 11 seed acceleration time histories that represent the contributing seismic sources to the total hazard at the site.
4. Modify the frequency content of the time histories from Step 2 via spectral matching to approximately match the target firm ground MCE UHS from Step 1.
5. Develop shear wave velocity profiles and one-dimensional (1D) soil models based on geotechnical explorations and geophysical measurements completed at the site.
6. Complete a nonlinear site-specific response analysis to compute ground surface response spectra and develop site-specific soil AFs.
7. Develop maximum component adjustment (MCA) factors and risk coefficients per ASCE 7-16.
8. Develop deterministic (MCE_R) ground motions per ASCE 7-16 by computing the 84^{th} percentile maximum direction deterministic response spectrum for the controlling earthquake scenario.
9. Develop probabilistic MCE_R ground motions by scaling the firm ground MCE UHS by the site-specific soil AFs, MCA factors, and risk coefficients.
10. Develop the site-specific MCE_R response spectrum as the lesser of the probabilistic and deterministic MCE_R ground motions and compare it to 80 percent of the ASCE 7-16 code-based MCE_R response spectrum.

Site-Specific Probabilistic Seismic Hazard Analysis

A site-specific PSHA was completed. Relevant seismic sources based on the 2014 United States Geological Survey (USGS) seismic source characterization (SSC) model were considered. The 2014 USGS SSC model contains seismic source characteristics and recurrence models developed by USGS for the 2014 update of the National Seismic Hazard Maps (Petersen et al. 2014).

Ground Motion Models

A suite of GMMs (Table E-1) was used to compute the ground motions from the three source-types: crustal, Cascadia Subduction Zone (CSZ) intraslab, and CSZ interface. Additional epistemic uncertainty per Al Atik and Youngs (2014) was included with the crustal GMMs to capture an appropriate level of epistemic uncertainty about the median and sigma models.

TABLE E-1. GROUND MOTION MODELS AND WEIGHTS

Earthquake Source-type	Ground Motion Model	Weight
Crustal	Abrahamson et al. (2014) [ASK14]	0.25
	Boore et al. (2014) [BSSA14]	0.25
	Campbell and Bozorgnia (2014) [CB14]	0.25
	Chiou and Youngs (2014) [CY14]	0.25
CSZ Intraslab	Atkinson and Boore – Global Subduction (2003, 2008) [AB08-G]	0.05
	Atkinson and Boore – Cascadia Subduction (2003, 2008) [AB08-C]	0.05
	Zhao et al. (2006) [Z06]	0.10
	BC Hydro – Base Global (Abrahamson et al. 2018)	0.48
	BC Hydro – Lower Global (Abrahamson et al. 2018)	0.16
	BC Hydro – Upper Global (Abrahamson et al. 2018)	0.16
CSZ Interface	Atkinson and Boore – Global Subduction (2003, 2008)	0.10
	Zhao et al. (2006)	0.10
	BC Hydro – Base Global (Abrahamson et al. 2018)	0.48
	BC Hydro – Lower Global (Abrahamson et al. 2018)	0.16
	BC Hydro – Upper Global (Abrahamson et al. 2018)	0.16

The GMMs and their associated epistemic weights represent the practicing state-of-art in ground-motion in the Pacific Northwest region. The NGA-West 2 GMMs (ASK14, BSSA14, CB14, CY14) developed in 2014 are the latest and most comprehensive GMMs published for crustal sources. The Abrahamson et al. (2014) [ASK14], Boore et al. (2014) [BSSA14], Campbell and Bozorgnia (2014) [CB14] and Chiou and Youngs (2014) [CY14] GMMs (Table E-1) were equally weighted.

The 2018 BC Hydro GMM is an update to the 2016 BC Hydro GMM (Abrahamson et al. 2016) and is applicable for both subduction-interface and subduction-intraslab source-types. It is based on a more comprehensive and up-to-date subduction zone ground motion database. The 2018 BC Hydro GMM consists of three epistemic branches: base, upper, and lower models. The base model represents the median ground motion. The upper and lower models represent the average of the upper- and lower-bound ground motions observed from the subduction zone ground motion dataset. In general, the 2018 BC Hydro GMM predicts slightly smaller ground motions than the 2016 BC Hydro GMM from intraslab source-type and predicts larger ground motions than the 2016 BC Hydro GMM from interface source-type at large distances (>300 kilometers [km]).

Unlike the 2016 version, the 2018 BC Hydro GMM does not include a regional variation for the Cascadia region. The developers found that in general the Cascadia ground motions were lower than the ground motions from other global subduction regions, but felt a regional model was not justified based on the small dataset size from the Cascadia region and lack of a physical basis for the ground motions being smaller. Alternatively, the 2018 BC Hydro GMM developers issued the upper and lower branch GMMs to represent the epistemic uncertainty in the ground motion predictions.

The 2018 BC Hydro base, upper and lower GMM, Zhao (2006) [Z06] and Atkinson and Boore (2003, 2008) [AB08] GMMs were selected to represent the GMMs for subduction-intraslab and subduction-interface source-types. The 2018 BC Hydro was assigned the largest epistemic weight of 0.80 because it represents the most state-of-the-art subduction GMM. The 2018 BC Hydro branch weight was then divided into three branch weights of 0.48, 0.16, and 0.16 to represent the base, upper and lower GMM, respectively. The Z06 and AB08 GMMs were given an epistemic weight of 0.1 each, which was deemed appropriate given the year the models were developed in comparison to the 2018 BC Hydro GMM.

Other GMMs were considered for this study but were not used. These GMMs include the crustal NGA-West 2 GMM by Idriss (2014) [I14], the subduction GMMs by Atkinson and Macias (2009) [AM09] and Zhao (2016) [Z16]. The I14 GMM was excluded because it does not include as many detailed modeling features as the other models used (e.g., the lack of distinction between normal and strike-slip fault mechanisms, hanging wall term, and basin depth term) (Petersen et al. 2014). The AM09 GMM was excluded because it results in a potentially unrealistic flatter spectral decay at long periods compared to other subduction-interface GMMs and it is based on numerical simulations. The Z16 was excluded because it also results in a flatter spectral decay.

Near-Field Directivity Effects

The site is located within 15 kilometers (km) of an active fault capable of producing moment magnitude (Mw) 7 or larger events (i.e., the Seattle fault zone and South Whidbey Island fault zone). Per ASCE 7-16 this is a near-field site, which may experience effects of ground motion rupture directivity. Directivity effects can be important to engineering structures because they can produce pulse-like motions for near-field sites (Shahi and Baker 2011).

Incorporation of rupture directivity effects into ground motion predictions started with the publication of the Somerville et al. (1997) and Abrahamson (2000) rupture directivity models. Since then several empirical ground motion rupture directivity models for crustal source-types have been published in the literature with the most recent models derived from the NGA-West2 Project (Spudich et al. 2014). Previous to, and with the publication of these directivity models, there has been a lack of clarity in practice regarding the application of these models (PEER 2015). To address this issue, PEER formed the PEER Directivity End-Users Panel project, which was tasked with developing recommendations for the implementation of directivity into ground motion predictions in the short-term. The report regarding their findings is pending. In the interim time, key findings from this Panel have been published in conference proceedings (Bozorgnia and Donahue 2016) and were utilized for this analysis. The most significant finding was that the Panel did not formally adopt a published directivity model, citing technical and implementation issues with each of the available models. Consequently, it was decided not to adopt any NGA-West2 rupture directivity models in our analysis as we believe further improvements in the models are required prior to adoption by the practicing community.

Similarly, rupture directivity effects were not included in the ground motion predictions of subduction zone source-types for this analysis. There are no known published subduction zone directivity models available at this time. This may change in the future with the NGA-Subduction Project that is currently underway.

In our opinion, potential unconservatism associated with not including directivity effects into the ground motion predictions is offset in our analysis by conservatively including the Al Atik and Youngs (2014) model of additional epistemic uncertainty in the ground motion logic tree and later adjusting the ground motion predictions to be maximum direction (RotD100) ground motions as required by ASCE-7-16.

Sedimentary Basin Effects

The site is located within the Seattle basin; therefore, it is critical to consider the potential amplification from sedimentary basin effects. Frankel et al. (2018) showed that the Seattle basin amplifies ground motion through four processes: (1) amplification and resonance of near-vertically propagating shear (S) waves, (2) conversion of incident S waves at the edge of the basin, (3) focusing of S waves by the edge of the basin, and (4) amplification of incoming surface waves.

The source-specific basin AFs were incorporated into the PSHA as ground motion scale factors computed from the following methodologies according to source-type:

1. **Crustal.** Semi-empirical crustal basin factors based on the depth to the shear wave velocity horizon of 2.5 kilometers per second (km/sec) ($Z_{2.5}$) computed per the CB14 NGA-West2 GMM. $Z_{2.5}$ is about 6.5 km per the Stephenson et al. (2017) velocity model.
2. **CSZ intraslab.** Average CSZ intraslab basin factors computed from recorded and processed ground motions from the M6.8 Nisqually and M6.4 Vancouver earthquakes presented by Chang et al. (2014) (Figure E-1).
3. **CSZ interface.** Recommended CSZ interface basin factors computed from three-dimensional (3D) simulations of M9.0 CSZ interface events (Frankel 2018) (Figure E-2). The computed basin factors (gray lines) were developed by taking the ratio of the average residual (i.e., comparing the simulated ground motions to the BC Hydro GMM) of all stations located within the basin (i.e., with $Z_{2.5} \geq 6.0$ km) to the average residual of multiple reference sites located outside the basin (with approximate $Z_{2.5}$ values of 3.0 km) (Wirth et al. 2018). Median and ±1 sigma basin AFs were developed based on 30 logic tree M9 earthquake scenarios. It was recognized that the reference $Z_{2.5}$ value of 3.0 km is approximate.

Figure E-3 and Table E-2 present basin factors for the approaches 1 through 3 listed above.

TABLE E-2. SOURCE-SPECIFIC BASIN AMPLIFICATION FACTORS

Period (sec)	Shallow Crustal	CSZ Intraslab	CSZ Interface
0.01	1.18	1.00[1]	1.00
0.05	1.13	1.03[1]	1.03
0.075	1.15	1.05[1]	1.04
0.1	1.18	1.06[1]	1.05
0.2	1.19	1.13[1]	1.10
0.3	1.26	1.19[1]	1.15

Period (sec)	Shallow Crustal	CSZ Intraslab	CSZ Interface
0.4	1.26	1.26[1]	1.20
0.5	1.32	1.32	1.25
0.75	1.41	1.39[2]	1.38
1	1.41	1.45	1.50
2	1.43	2.12	2.00
3	1.40	2.45	2.00
4	1.42	2.08	2.00
5	1.43	1.93	2.00
7.5	1.56	1.65	2.00
10	1.48	1.30	2.00

Notes:

[1] linearly extrapolated from T=0.5 seconds to T=0 seconds (peak ground acceleration [PGA])

[2] linearly interpolated between T=0.5 and T=1.0 seconds

Probabilistic MCE Ground Motions

Figure E-4 and Table E-3 present the site-specific 5 percent damped basin-adjusted firm ground MCE UHS.

TABLE E-3. BASIN-ADJUSTED FIRM GROUND MCE UHS

Period (sec)	5% Damped Spectral Acceleration, Sa (g)
0.01	0.630
0.05	0.911
0.075	1.203
0.1	1.454
0.2	1.598
0.3	1.373
0.4	1.137
0.5	1.003
0.75	0.726
1	0.557
2	0.304
3	0.184
4	0.118
5	0.088

Site-Specific Response Analysis

Nonlinear site-specific response analysis (SSRA) was completed using Fast Lagrangian Analysis of Continua (FLAC) (Itasca 2016). The purpose of the analysis is to compute site-specific soil AFs. The following subsections describe our analysis approach and results.

Ground Motion Selection

Per email correspondence with the project structural engineer (Magnusson Klemencic Associates [MKA]) dated September 6, 2018, the preliminary fundamental period of the buildings is estimated to be between about 0.5 and 1.0 second. Seismic hazard deaggregation was performed at spectral periods of 0.5, 0.75, and 1.0 second to compute the percent contribution of the various source-types to the total MCE hazard. The percent contribution results are presented in Table E-4. Table E-5 presents the number of seed ground motions selected to represent each source-type. The crustal sources are subdivided between near-field and far-field sources based on the source-to-site distance for each discretely mapped fault. The Seattle fault zone and South Whidbey Island fault zone are classified as near-field sources and are represented by pulse-like crustal ground motions. The far-field crustal sources are represented by far-field crustal ground motions. Table E-6 presents characteristics of the suite of records selected for the MCE.

TABLE E-4. PERCENT CONTRIBUTION RESULTS (V_{S30}=701 M/SEC)

Earthquake Source	Percent Contribution		
	T=0.50 sec	T=0.75 sec	T=1.0 sec
Seattle Fault Zone (near-field)	7	8	8
South Whidbey Island Fault Zone (near-field)	13	14	14
Gridded Background Seismicity (far-field)	39	41	35
Other Crustal Faults (far-field)	0	0	0
CSZ Intraslab	17	10	8
CSZ Interface	24	27	35

TABLE E-5. RECOMMENDED GROUND MOTION DISTRIBUTION

Earthquake Source-type	Number of Records
Crustal (near-field)	3
Crustal (far-field)	4
CSZ Intraslab	1
CSZ Interface	3
Total	**11**

TABLE E-6. CHARACTERISTICS OF INPUT EARTHQUAKE TIME HISTORIES

Record ID	Earthquake	Source Mechanism	Mw	Station	Component	Distance (km)	Near-field Record (Y/N)
GM-1	Chuetsu-Oki, Japan 2007	Crustal (Reverse)	6.8	Kawaguchi	NS	29	N
GM-2	Loma-Prieta, CA 1989	Crustal (Reverse Oblique)	6.9	San Jose - Santa Teresa Hills	225	15	N
GM-3	Iwate, Japan 2008	Crustal (Reverse)	6.9	Yamauchi Tsuchibuchi Yokote	NS	29	N

Record ID	Earthquake	Source Mechanism	Mw	Station	Component	Distance (km)	Near-field Record (Y/N)
GM-4	San Simeon, CA, 2003	Crustal (Reverse)	6.5	Cambria - Hwy 1 Caltrans Bridge	090	7	N
GM-5	Loma Prieta, CA, 1989	Crustal (Reverse Oblique)	6.9	Gilroy Array #3	Fault Normal	13	Y
GM-6	Northridge, 1994	Crustal (Reverse)	6.7	Newhall - Fire Station	Fault Normal	6	Y
GM-7	Chuetsu-Oki, Japan 2007	Crustal (Reverse)	6.8	Joetsu Kakizakiku Kakizaki	Fault Normal	12	Y
GM-8	Olympia, WA, 1949	Subduction-intraslab	6.8	WSDOT Highway Test Lab	086	75	N
GM-9	Tohoku, 2011	Subduction-interface	9.0	Ujiie TCGH12	EW	299	N
GM-10	Maule, 2010	Subduction-interface	8.8	Concepcion San Pedro	NS	36	N
GM-11	Tohoku, 2011	Subduction-interface	9.0	Onoda MYGH05	EW	189	N

Ground Motion Modification

The seed ground motions were modified via spectral matching to match the target firm ground MCE UHS from T=0.01 to 5.0 seconds. Spectral matching was completed using RSPMatch09 (Fouad et al. 2012) based on the improved spectral matching approach proposed by Al Atik et al. (2010). The ground motions were processed with a Butterworth low pass filter to filter out frequencies greater than 25 Hertz (Hz). The as-recorded and spectrally matched response spectra are presented in Figures E-5 and E-6. Figures E-7 through E-17 present similar figures for the acceleration, velocity, and displacement time histories.

One-dimensional Soil Models

Shear wave velocity (Vs) profiles were developed based on the results of site-specific geophysical measurements.

Four 1D soil models were developed (two for the north building and two for the south building) and are shown in Figures E-18A and E-18B for the shallow and deep profiles, respectively. Tables E-7A through E-7D summarize the soil properties and shear modulus reduction (G/Gmax) and damping curves used in the 1D soil models. The soil models were discretized so that the mesh is capable of transmitting frequencies up to 25 Hz.

TABLE E-7A. ONE-DIMENSIONAL SOIL MODEL, NORTH-EAST

Depth Range (feet)	Shear Wave Velocity (ft/sec)	Unit Weight (pcf)	Hysteretic Damping Model[1]
0 to 6	726	125	1
6 to 9	770	125	1
12 to 23	840	130	1

Depth Range (feet)	Shear Wave Velocity (ft/sec)	Unit Weight (pcf)	Hysteretic Damping Model[1]
23 to 50	950	130	1
50 to 57	950	130	2
57 to 61	1,025	130	2
61 to 123	1,170	130	2
123 to 198	1,400	130	2
198 to 373	1,850	130	2
373 to 584	2,020	130	2
584 to 1112	2,060	130	2
1,112 to 1,428	2,150	130	2
1,428 to 1,942	2,250	130	2
below 1,942	2,300	130	2

Note:
[1] Damping Models: (1) Darendeli [PI=0], (2) Silva Peninsular Range [50'-1000']
pcf – pounds per cubic foot

TABLE E-7B. ONE-DIMENSIONAL SOIL MODEL, NORTH-WEST

Depth Range (feet)	Shear Wave Velocity (ft/sec)	Unit Weight (pcf)	Hysteretic Damping Model[1]
0 to 2	770	130	1
2 to 4	830	130	1
4 to 6	970	130	1
6 to 9	1,080	130	1
9 to 14	1,150	130	1
14 to 18	1,230	130	1
18 to 26	1,330	130	1
26 to 50	1,390	130	1
50 to 87	1,390	130	2
87 to 198	1,440	130	2
198 to 374	1,850	130	2
374 to 586	2,010	130	2
586 to 1,112	2,050	130	2
1,112 to 1,428	2,140	130	2
1,428 to 1,942	2,250	130	2
below 1,942	2,280	130	2

Note:
[1] Damping Models: (1) Darendeli [PI=0]], (2) Silva Peninsular Range [50'-1000']
pcf – pounds per cubic foot

TABLE E-7C. ONE-DIMENSIONAL SOIL MODEL, SOUTH-EAST

Depth Range (feet)	Shear Wave Velocity (ft/sec)	Unit Weight (pcf)	Hysteretic Damping Model[1]
0 to 4	785	125	1
4 to 6	965	125	1
6 to 9	1,110	130	1
9 to 12	1,180	130	1
12 to 14	1,250	130	1
14 to 18	1,340	130	1
18 to 50	1,410	130	1
50 to 61	1,410	130	2
61 to 198	1,620	130	2
198 to 281	1,770	130	2
281 to 374	1,900	130	2
374 to 585	2,040	130	2
585 to 1,430	2,090	130	2
1,430 to 1,942	2,230	130	2
below 1,942	2,300	130	2

Note:

[1] Damping Models: (1) Darendeli [PI=0], (2) Silva Peninsular Range [50'-1000']

pcf – pounds per cubic foot

TABLE E-7D. ONE-DIMENSIONAL SOIL MODEL, SOUTH-WEST

Depth Range (feet)	Shear Wave Velocity (ft/sec)	Unit Weight (pcf)	Hysteretic Damping Model[1]
0 to 7	830	125	1
7 to 14	920	125	1
14 to 31	1,290	130	1
31 to 50	1,460	130	1
50 to 57	1,460	130	2
57 to 198	1,620	130	2
198 to 281	1,780	130	2
281 to 374	1,900	130	2
374 to 1,429	2,070	130	2
1,429 to 1,942	2,230	130	2
below 1,942	2,300	130	2

Note:

[1] Damping Models: (1) Darendeli [PI=0], (2) Silva Peninsular Range [50'-1000']

pcf – pounds per cubic foot

Site-Specific Soil Amplification Factors

The suite of 11 horizontal time-histories were propagated upward through each 1D soil model (Table E-7A to E-7D) in FLAC and ground surface response spectra were computed. Four sets of soil AFs were then developed as the average of the ratio of the ground surface response spectra to the input firm ground response spectra for the ground motion suite. The mean AFs from the north profiles were averaged together and the mean AFs from the south profiles were averaged together to develop two sets of AFs to represent the expected ground amplification at the site for the MCE.

Figures E-19 through E-22 present the individual and average soil MCE AFs for the north-west, north-east, south-west and south-east profiles, respectively. Figure E-23 presents a summary comparison of the average MCE soil AFs. Figure E-24 presents the recommended MCE soil AFs for the north and south sides of the project site.

Maximum Component Adjustment Factors and Risk Coefficients

Per ASCE 7-16, the design ground motions are to be taken in the direction of maximum horizontal response. MCA factors convert the geometric mean spectral ordinates to spectral ordinates that correspond to the direction of maximum horizontal response. The MCA factors per Shahi and Baker (2014) were used for this evaluation.

Risk coefficients convert the probabilistic MCE ground motions to risk-targeted ground motions (MCE_R), which correspond to a 1 percent probability of collapse in 50 years. Risk coefficients were calculated per ASCE 7-16, Section 21.2.1.2. Table E-8 presents the MCA factors and site-specific risk coefficients.

TABLE E-8. MAXIMUM COMPONENT ADJUSTMENT FACTORS AND RISK COEFFICIENTS

Period (sec)	Maximum Component Adjustment Factor	Risk Coefficient
0.01	1.19	0.92
0.05	1.19	0.92
0.075	1.19	0.93
0.1	1.19	0.93
0.2	1.21	0.93
0.3	1.22	0.92
0.4	1.23	0.91
0.5	1.23	0.91
0.75	1.24	0.91
1	1.24	0.90
2	1.24	0.89
3	1.25	0.89
4	1.26	0.89
5	1.26	0.89

Deterministic MCE_R Ground Motions

Eighty-fourth percentile deterministic (MCE_R) ground motions were developed per ASCE 7-16 Section 21.2.2. The controlling deterministic scenario at the site is a Mw 7.4 earthquake located approximately 5 km away on the Whidbey fault zone. The suite of GMMs and weights (Table E-1) that were used to complete the PSHA were used to compute the deterministic response spectrum (84th percentile) for the average Vs30 value (about 1,264 feet per second). Basin amplification was accounted with crustal basin AFs (Table E-2). MCA factors (Table E-8) were used to convert the geometric mean deterministic ground motions to maximum direction round motions per ASCE 7-16, Section 21.2.2. Figure E-25 presents the deterministic MCE_R response spectrum.

Recommended Site-Specific MCE_R Response Spectra

The site-specific probabilistic MCE_R response spectrum was computed by scaling the probabilistic basin-adjusted firm ground MCE UHS by the site-specific MCE soil AFs, MCA factors, and risk coefficients. The site-specific MCE_R ground motions are taken as the lesser of the probabilistic MCE_R and deterministic MCE_R ground motions. Figure E-26 presents a comparison of the probabilistic and deterministic MCE_R response spectra, the ASCE 7-16 Site Class D MCE_R response spectrum, and 80 percent of the Site Class D MCE_R response spectrum (the allowable lower limit) for the north building. The north building site is classified as Site Class F per ASCE 7-16 due to the presence of liquefiable soils; however, ASCE 7-16 permits a site class other than E to be used in establishing the lower limit when a site is classified as F. This revision eliminates the possibility of an overly conservative design spectrum on sites such as this one that would normally be classified as Site Class C or D in the absence of liquefiable soils.

The recommended site-specific MCE_R response spectrum (solid blue line) is controlled by the probabilistic MCE_R in the spectral period range of 0.85 to 0.6 seconds and is controlled by the ASCE 7-16 minimum at all other periods.

Figure E-27 presents a similar comparison of the probabilistic and deterministic MCE_R response spectra, the ASCE 7-16 Site Class C MCE_R response spectrum, and 80 percent of the Site Class C MCE_R response spectrum (the allowable lower limit), but for the south building. The recommended site-specific MCE_R response spectrum (solid blue line) is controlled by the probabilistic MCE_R in the spectral periods greater than 0.85 seconds and is controlled by ASCE 7-16 at all other periods.

Tables E-9A and E-9B presents the recommended site-specific MCE_R response spectra for the north (Site Class F) and south (Site Class C) locations of the project site.

TABLE E-9A. RECOMMENDED SITE-SPECIFIC MCE_R RESPONSE SPECTRUM
NORTH, SITE CLASS F

Period (sec)	5% Damped Spectral Acceleration, Sa (g)
0.01	0.439
0.05	0.701
0.075	0.789
0.1	0.890
0.2	1.110
0.3	1.120

Period (sec)	5% Damped Spectral Acceleration, Sa (g)
0.4	1.100
0.5	1.090
0.6	1.010
0.87	1.010
1	0.882
2	0.441
3	0.294
4	0.221
5	0.176

TABLE E-9B. RECOMMENDED SITE-SPECIFIC MCE_R RESPONSE SPECTRUM SOUTH, SITE CLASS C

Period (sec)	5% Damped Spectral Acceleration, Sa (g)
0.01	0.568
0.05	0.997
0.075	1.149
0.09	1.212
0.44	1.212
0.5	1.06
0.6	0.88
0.75	0.76
0.8	0.735
0.9	0.667
1	0.596
1.3	0.477
2	0.321
3	0.217
4	0.152
5	0.115

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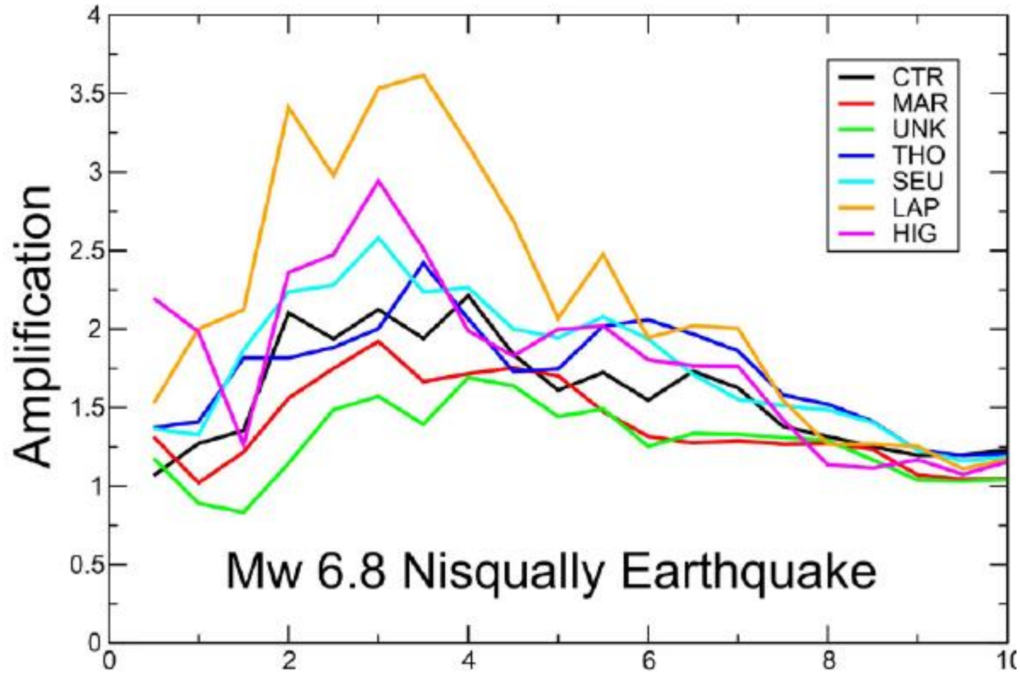

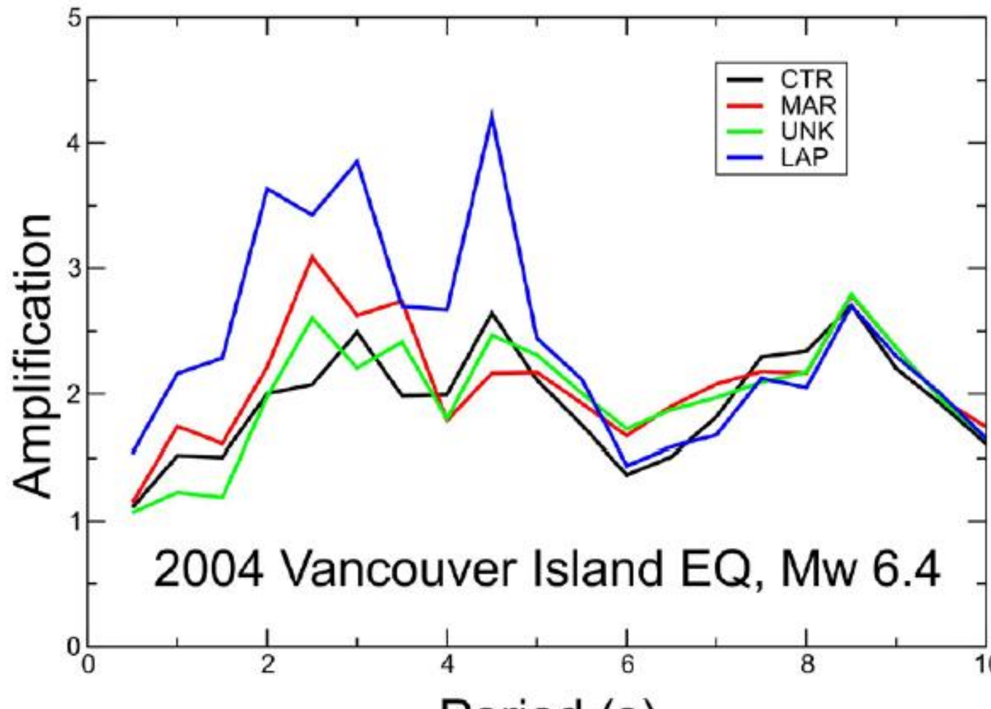

Reference: Chang et al. (2014)

Cascadia Subduction Zone Intraslab Basin Amplification Factors

Building X
Redmond, Washington

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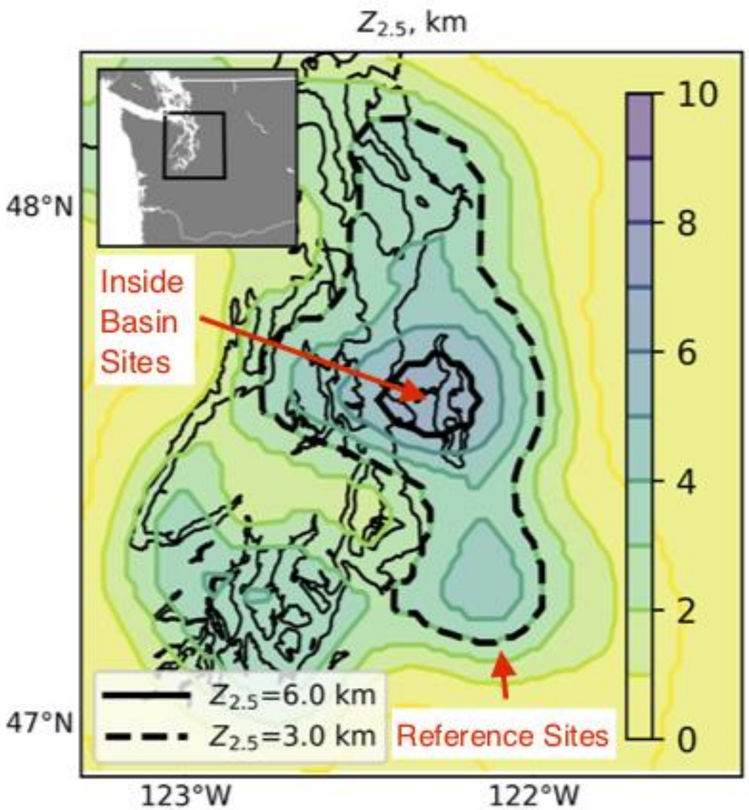
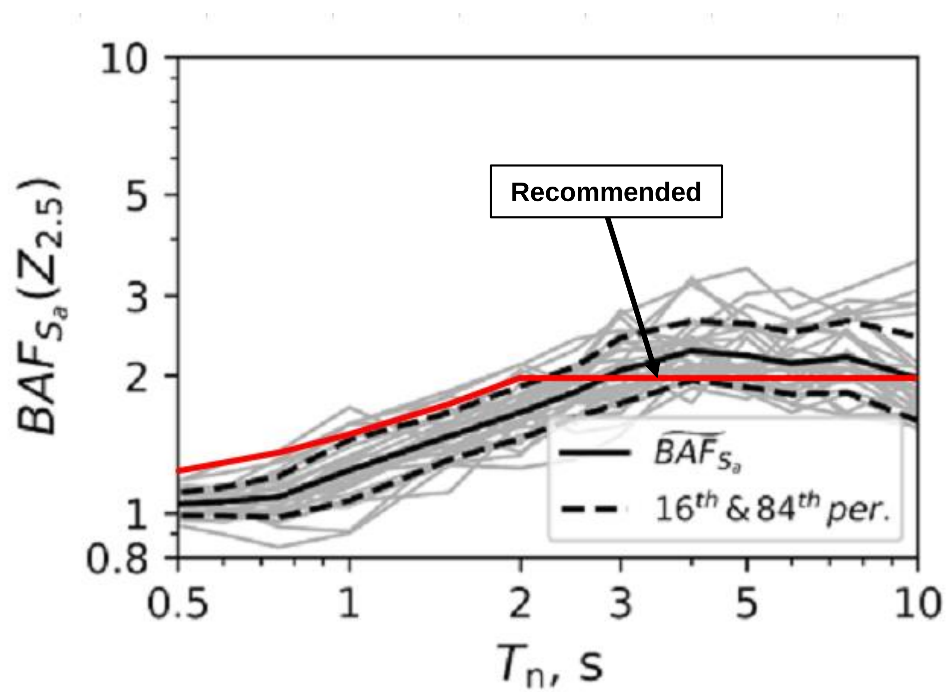
Figure E-1





Cascadia Subduction Zone Interface Basin Amplification Factors

Building X
Redmond, Washington

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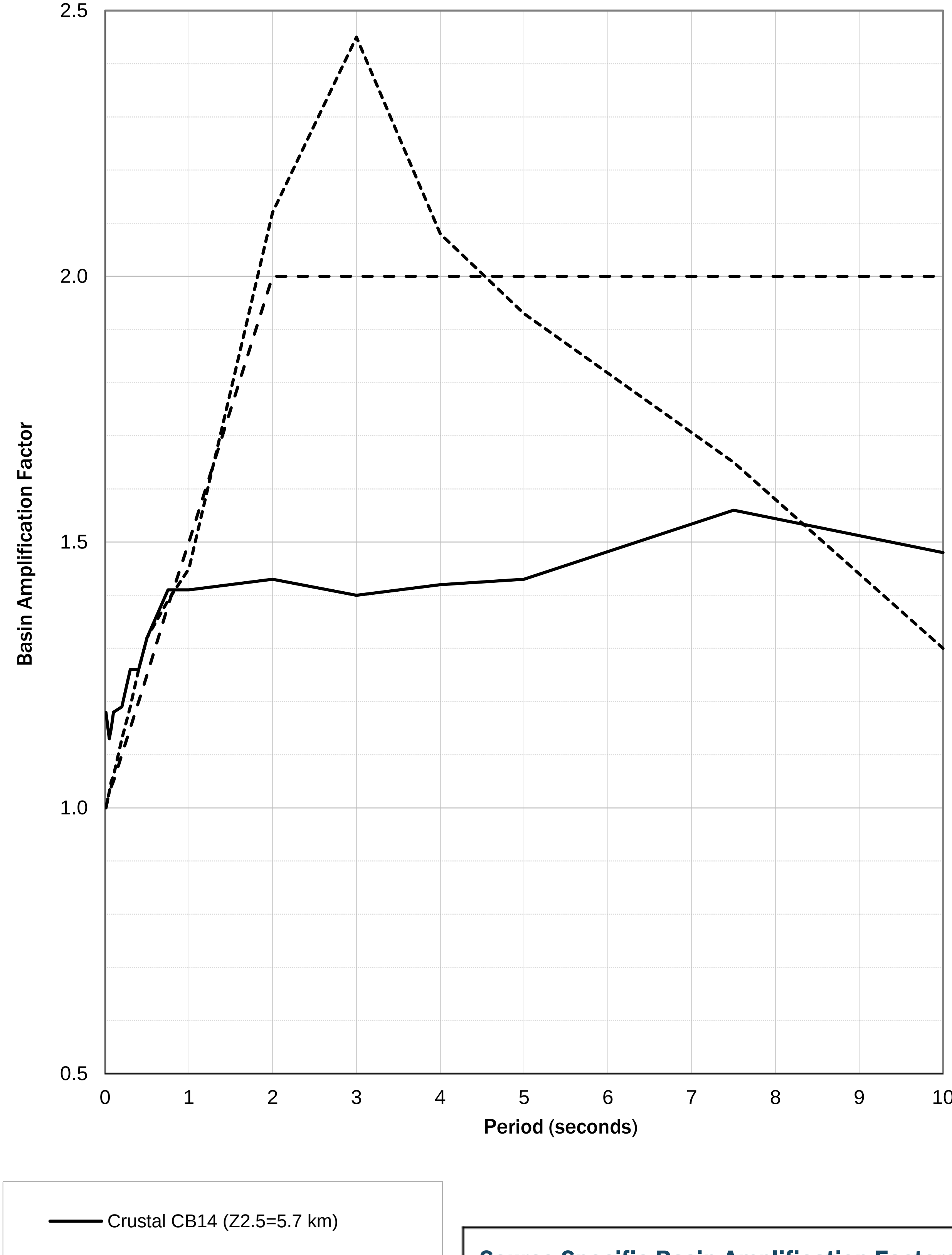
Figure E-2

Reference: Frankel et al. (2018); Marafi (2018)



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Source-Specific Basin Amplification Factors

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Redmond, Washington

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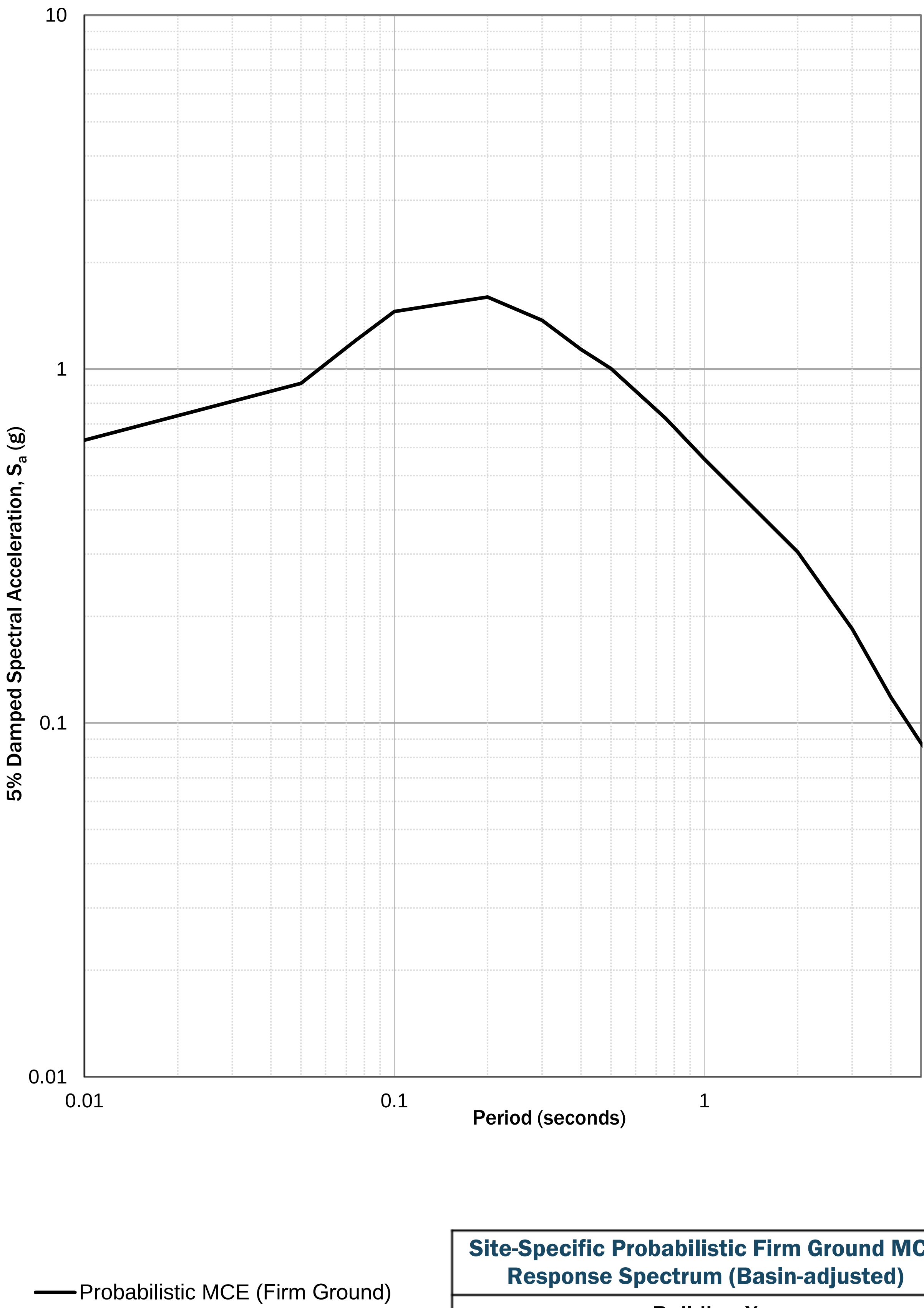
Figure E-3

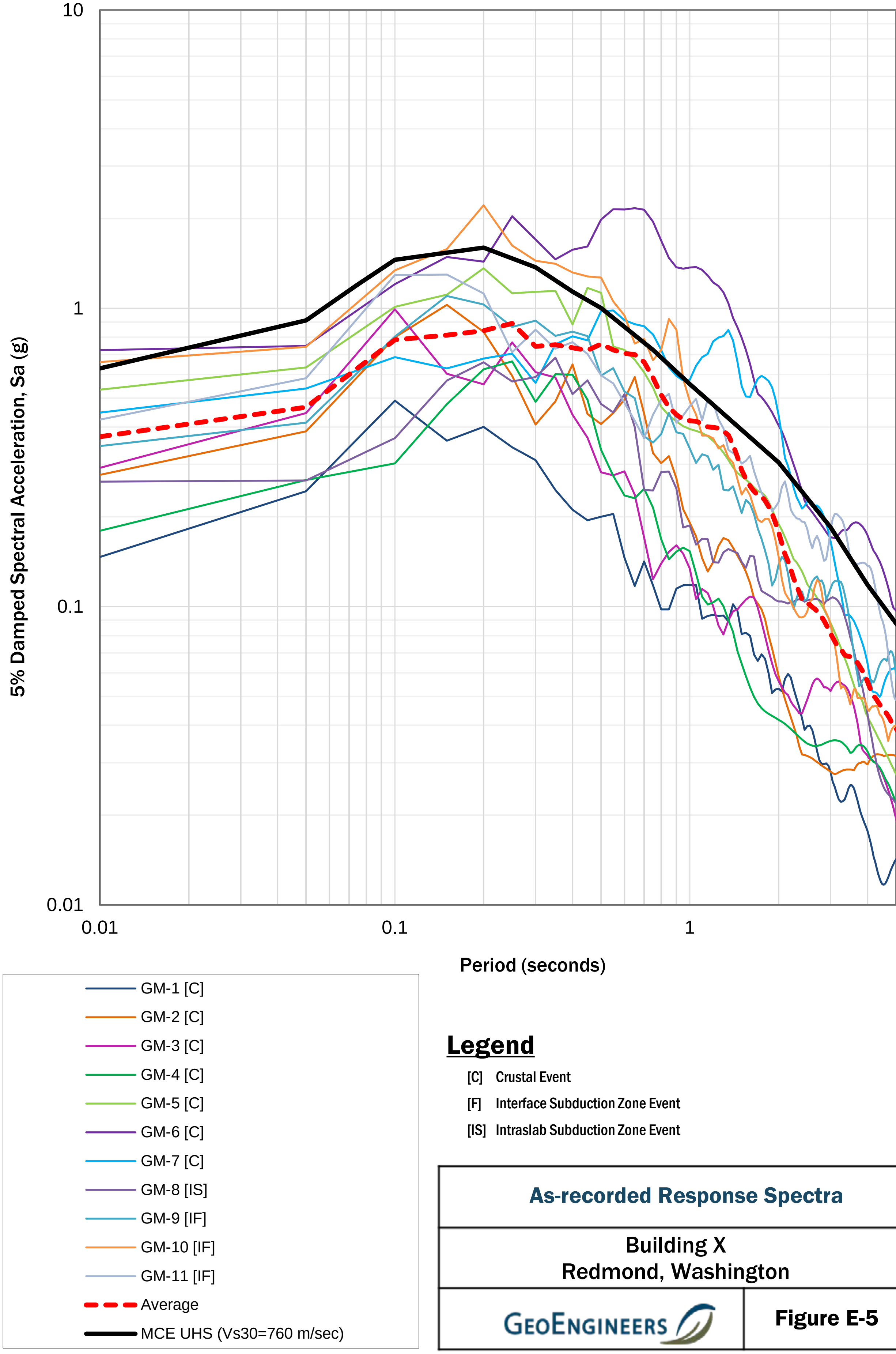


Site-Specific Probabilistic Firm Ground MCE Response Spectrum (Basin-adjusted)

Building X
Redmond, Washington

GeoEngineers

Figure E-4



Legend

[C] Crustal Event

[F] Interface Subduction Zone Event

[IS] Intraslab Subduction Zone Event

As-recorded Response Spectra	
Building X Redmond, Washington	
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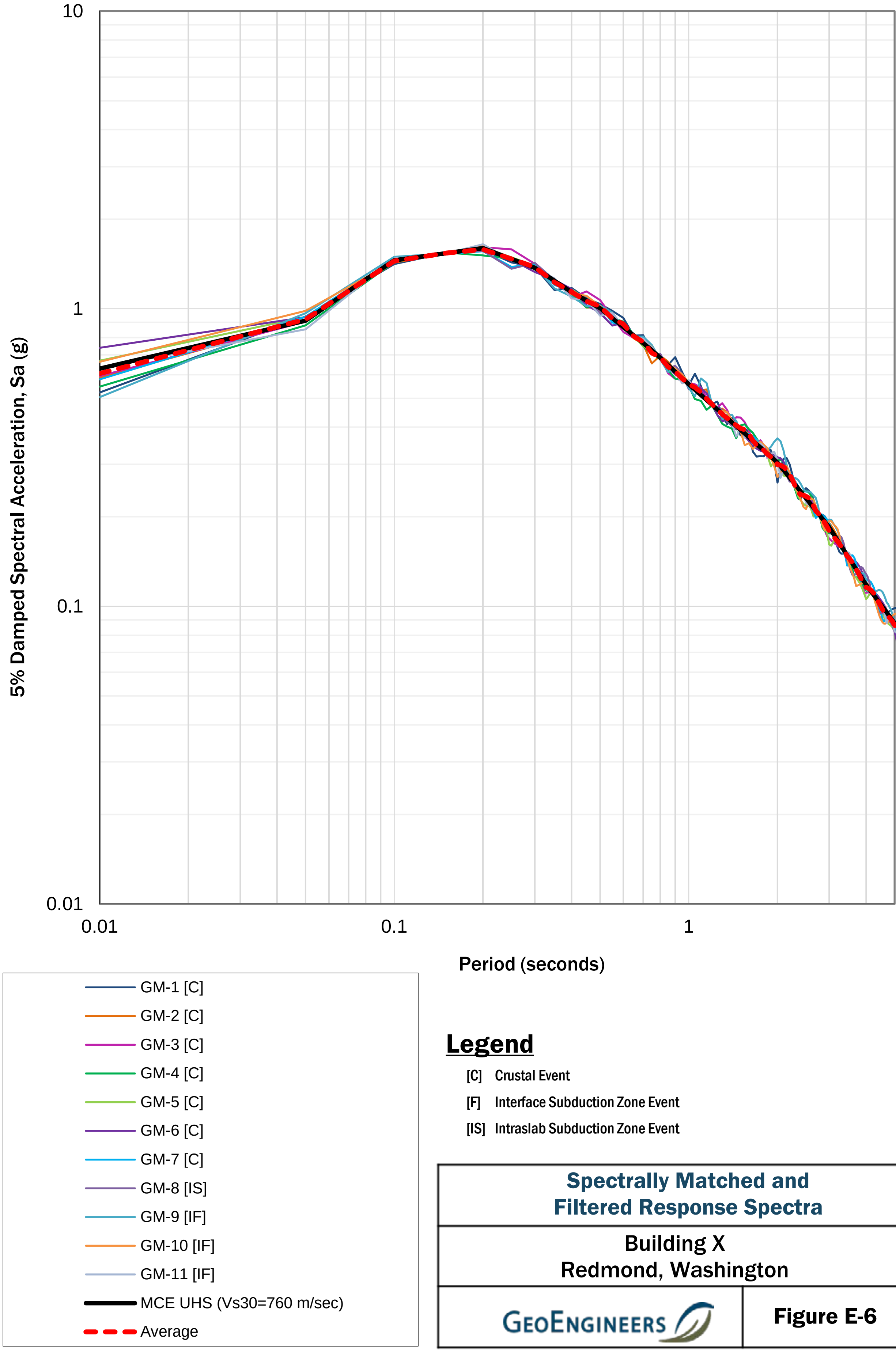


Legend

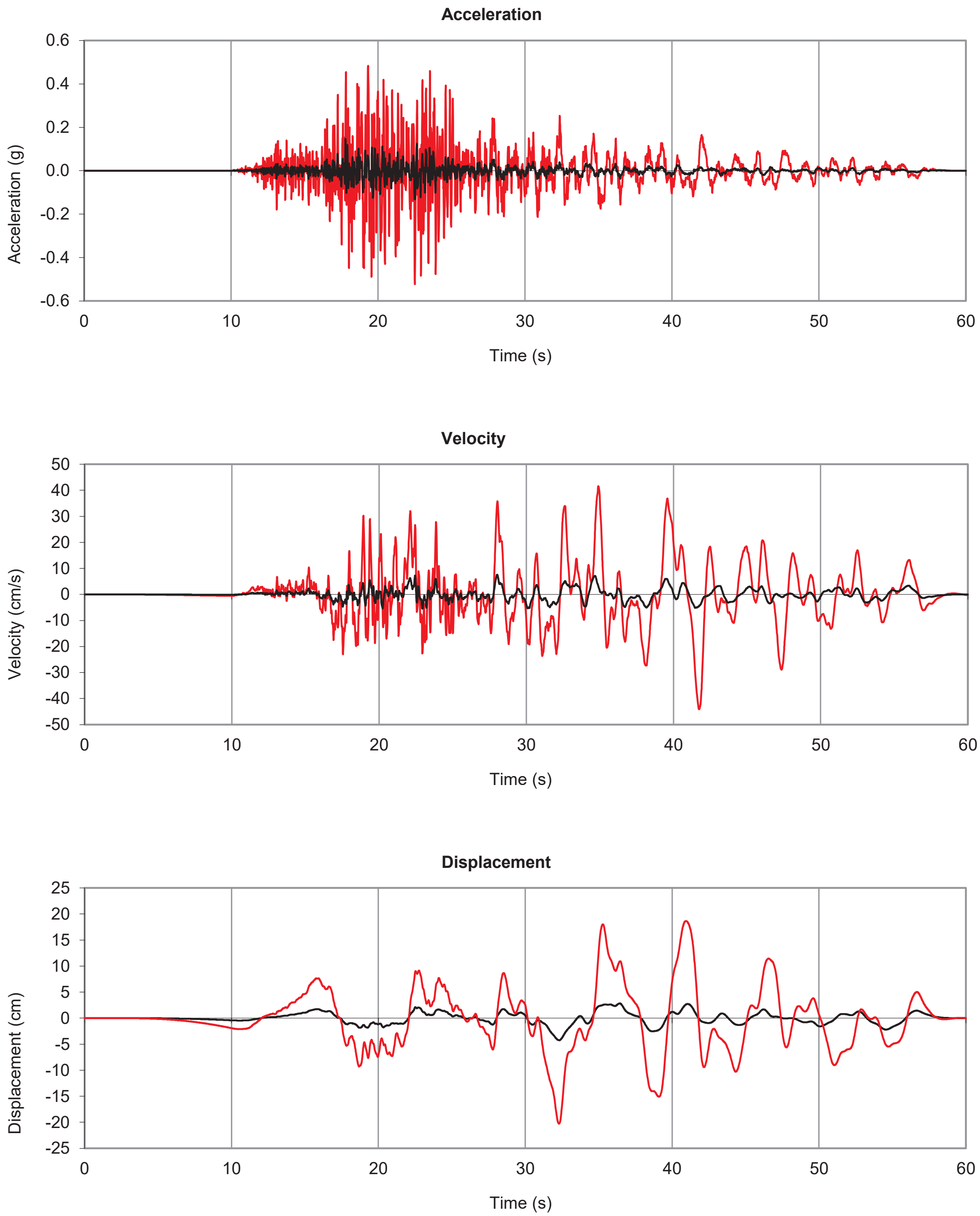
[C] Crustal Event

[F] Interface Subduction Zone Event

[IS] Intraslab Subduction Zone Event

Spectrally Matched and Filtered Response Spectra	
Building X Redmond, Washington	
GEOENGINEERS	Figure E-6

Acceleration, Velocity, and Displacement Time Series



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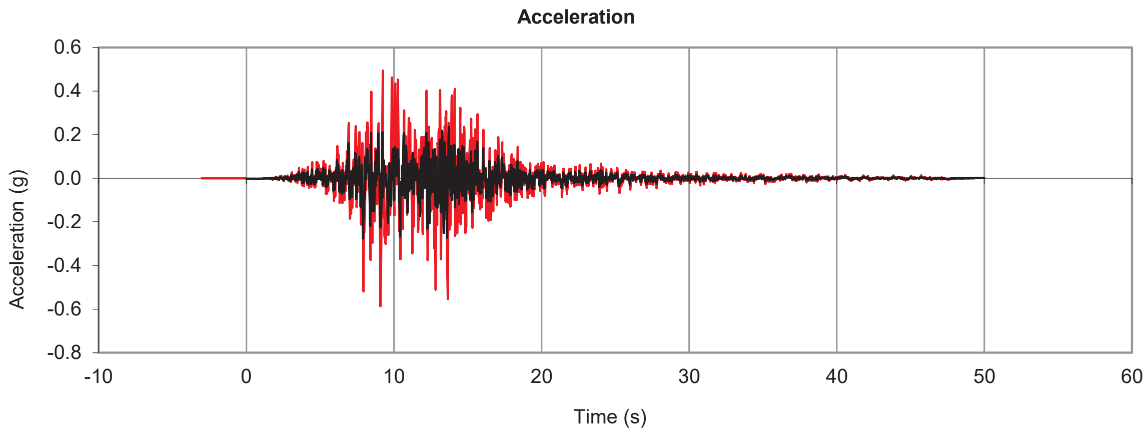
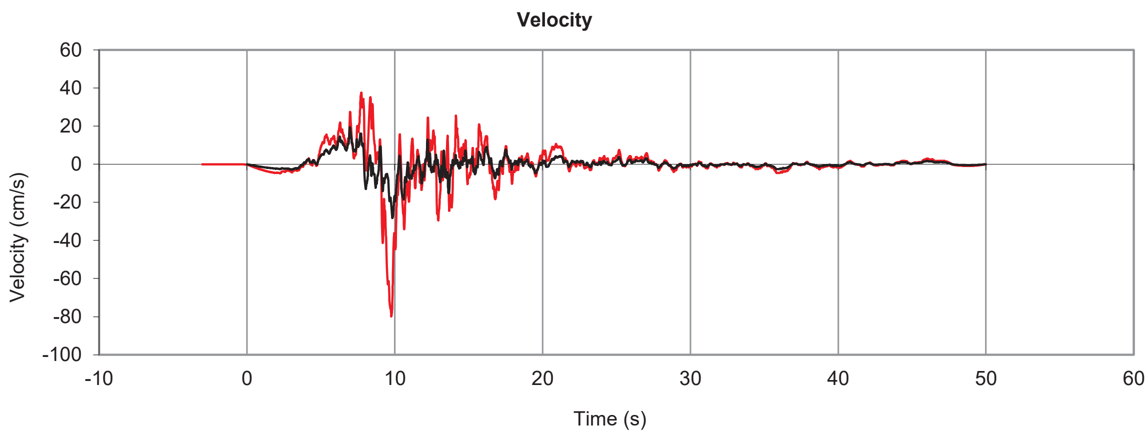
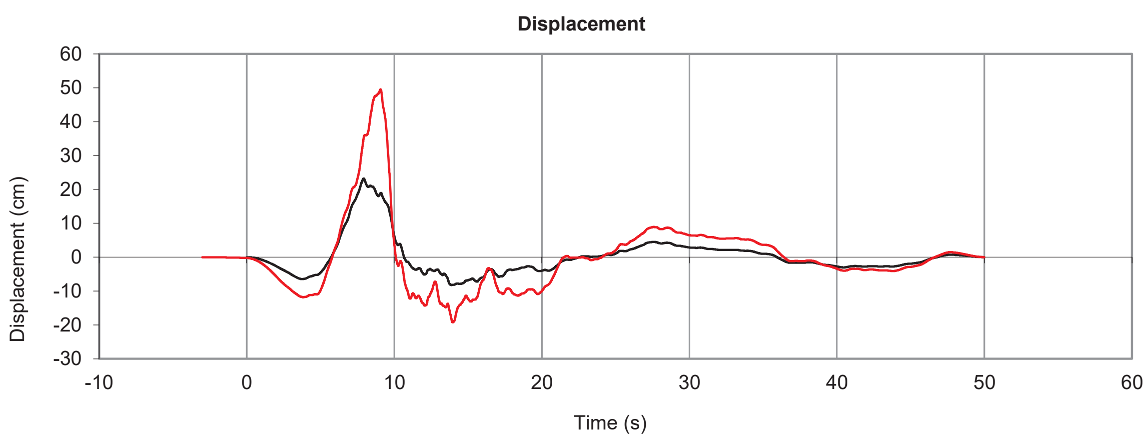
Chuetsu-Oki - RSN4896 - NS (GM-1)
Building X - Redmond, Washington

Figure E-7

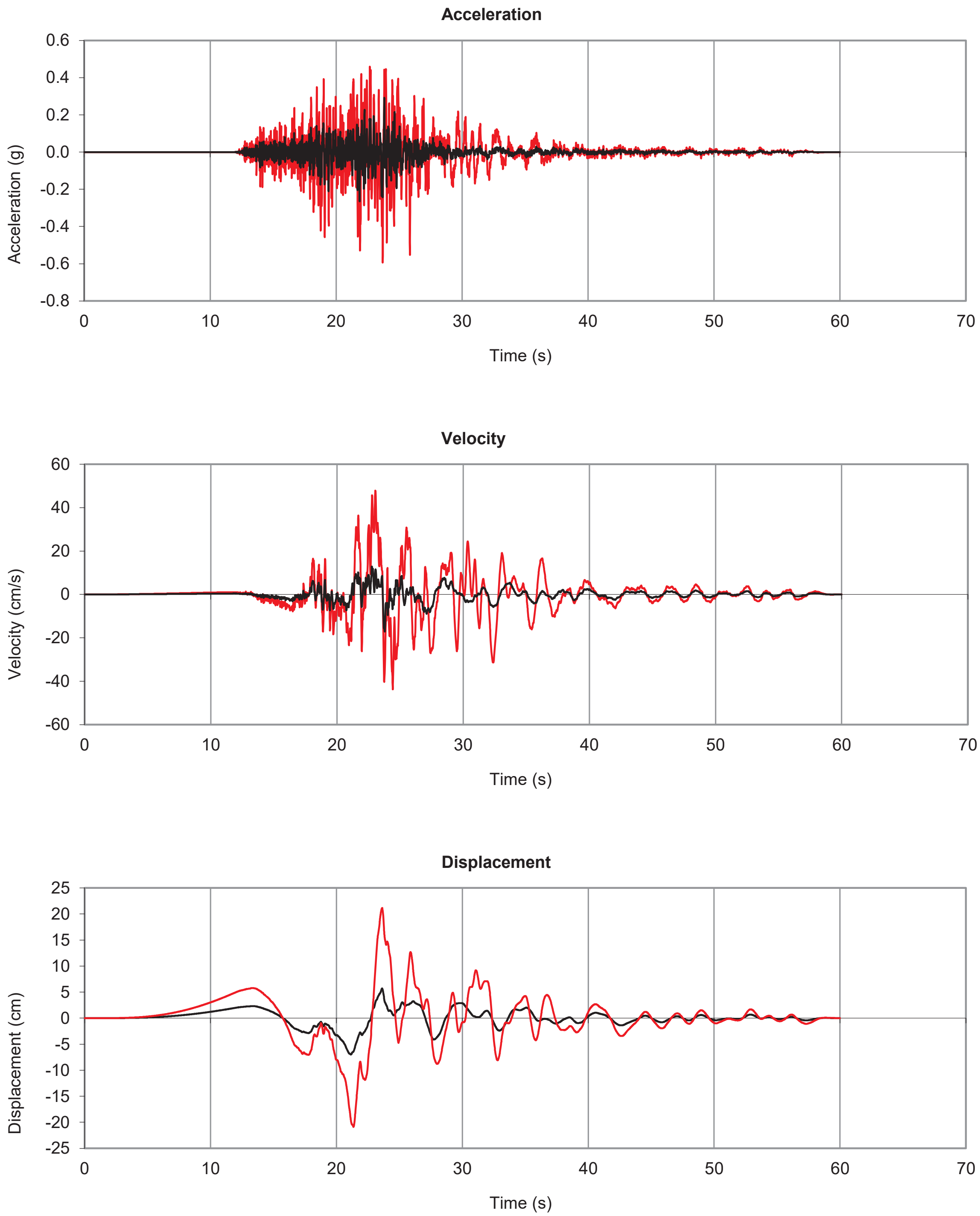
Acceleration, Velocity, and Displacement Time Series







Acceleration, Velocity, and Displacement Time Series



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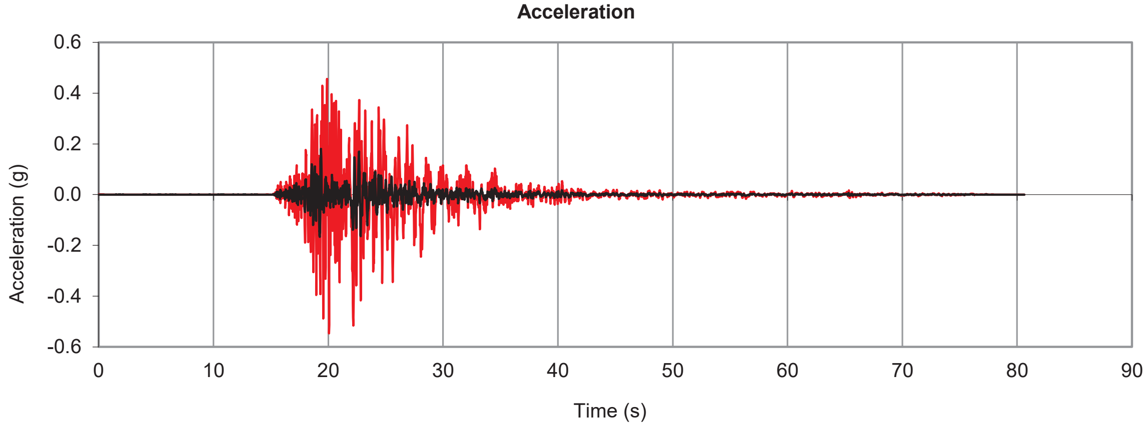
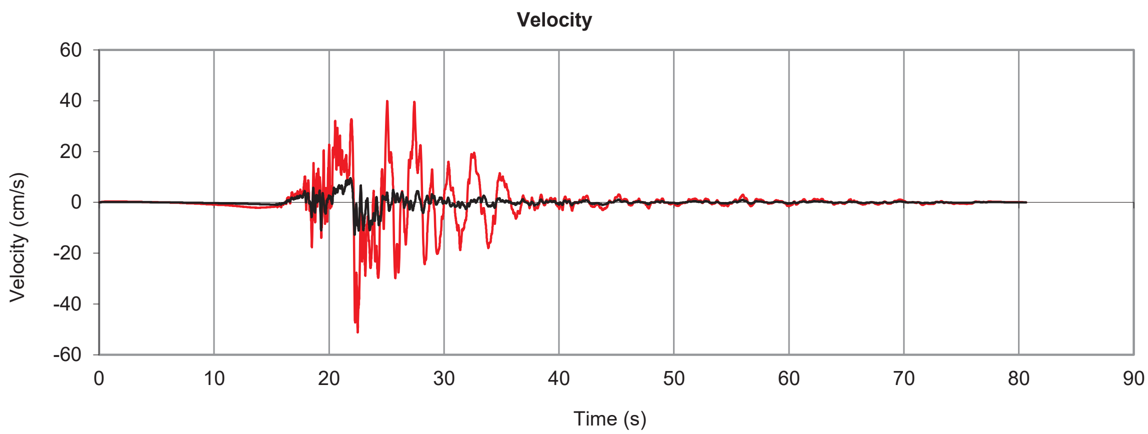
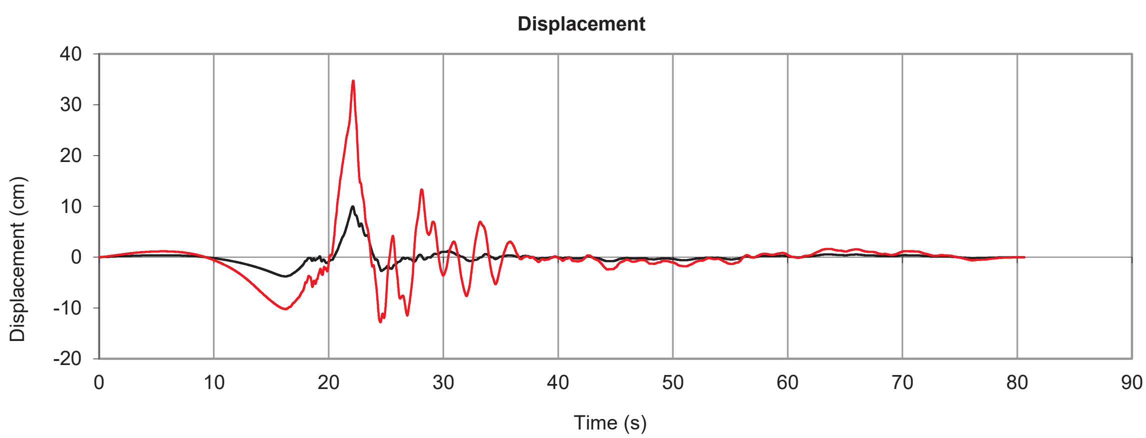
Iwate - RSN5804 - NS (GM-3)
Building X - Redmond, Washington

Figure E-9

Acceleration, Velocity, and Displacement Time Series







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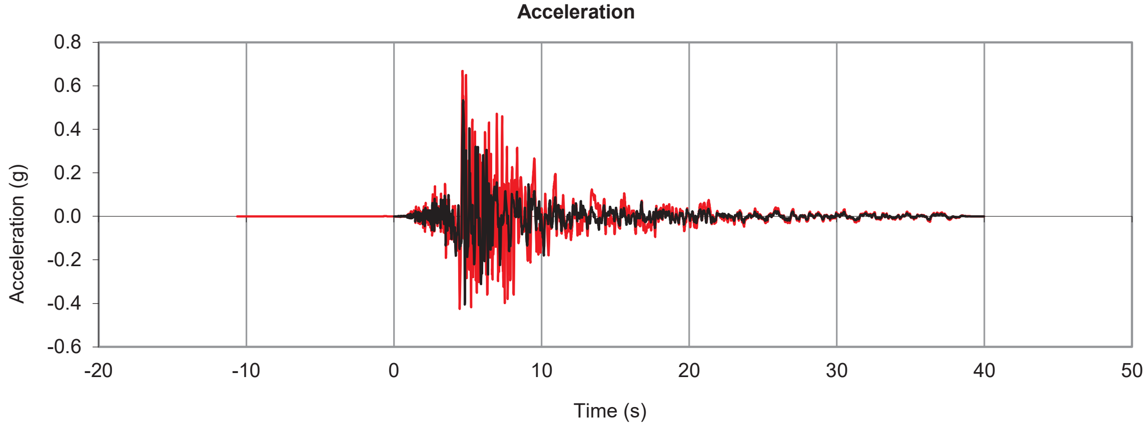
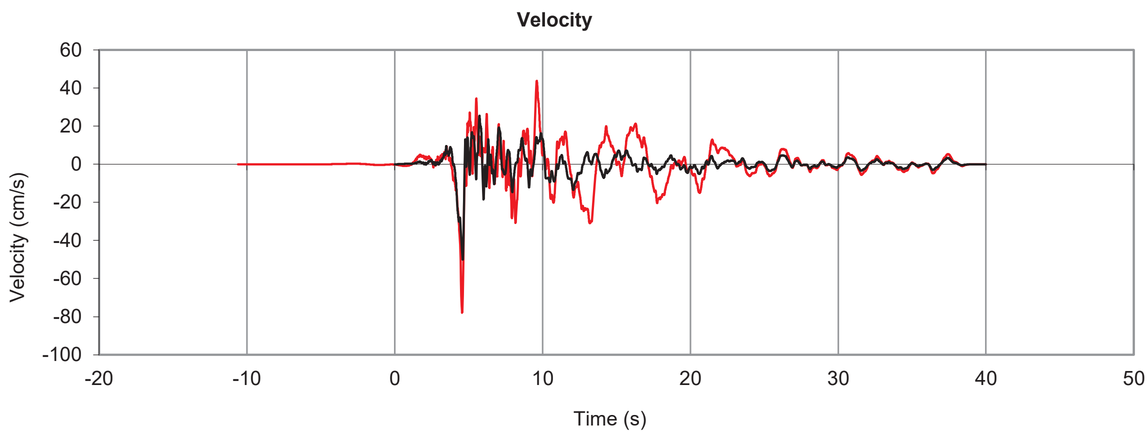
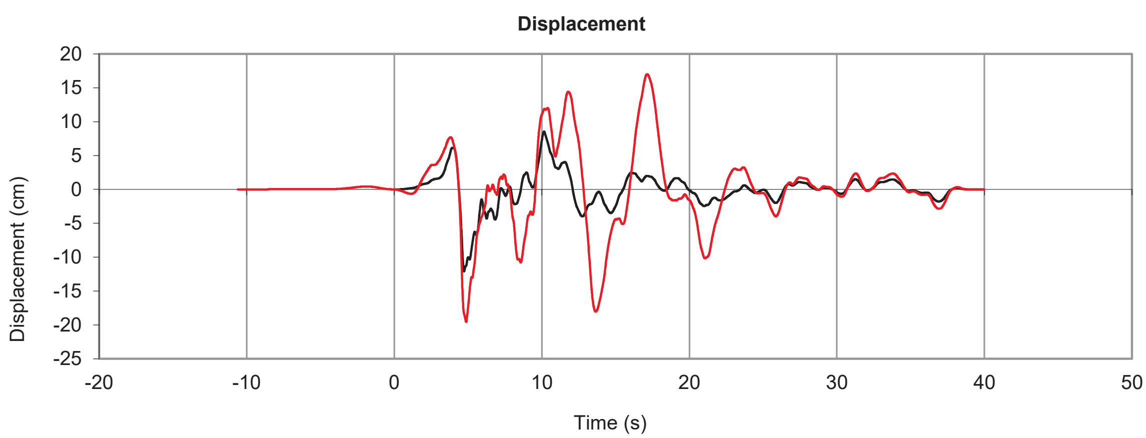
San Simeon - RSN397 - 090 (GM-4)
Building X - Redmond, Washington

Figure E-10

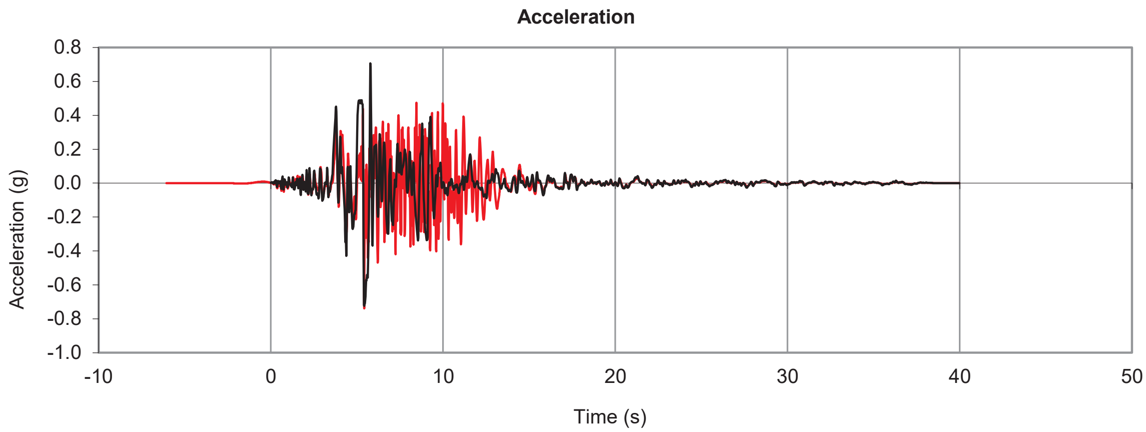
Acceleration, Velocity, and Displacement Time Series

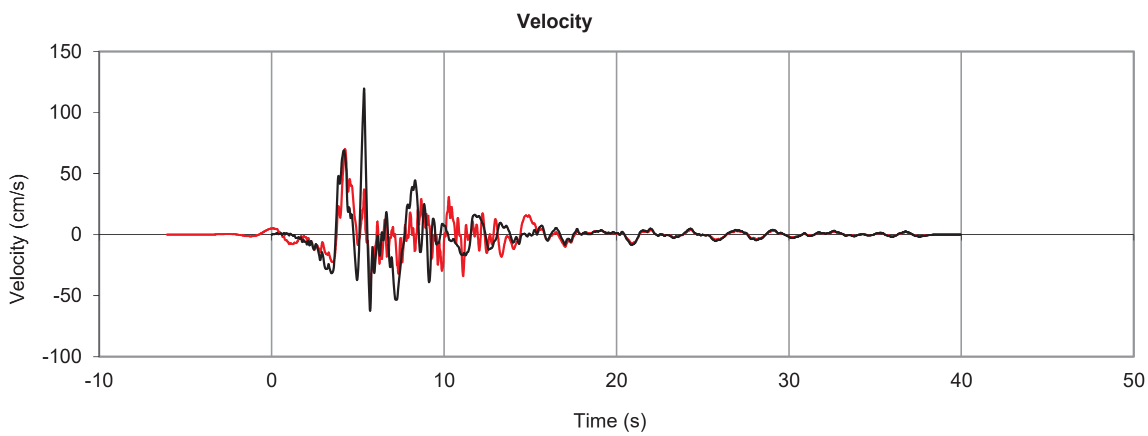


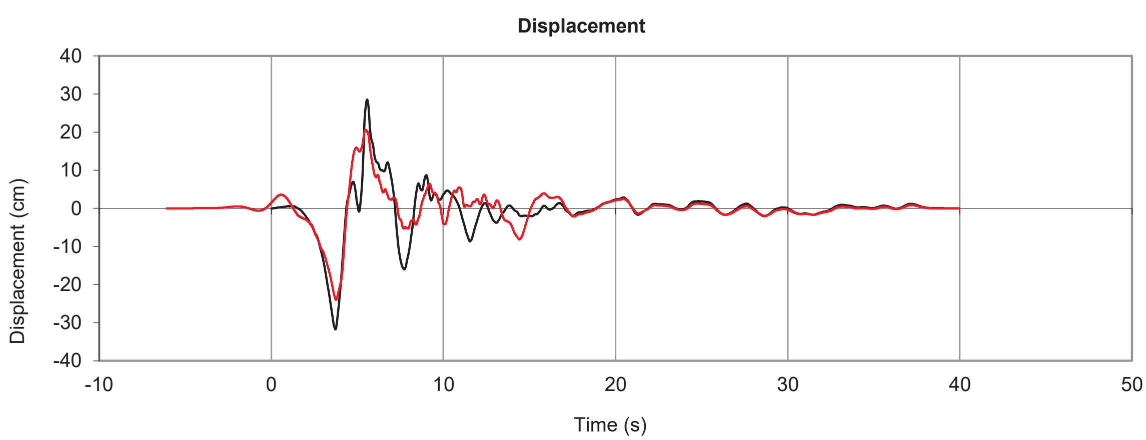




Acceleration, Velocity, and Displacement Time Series







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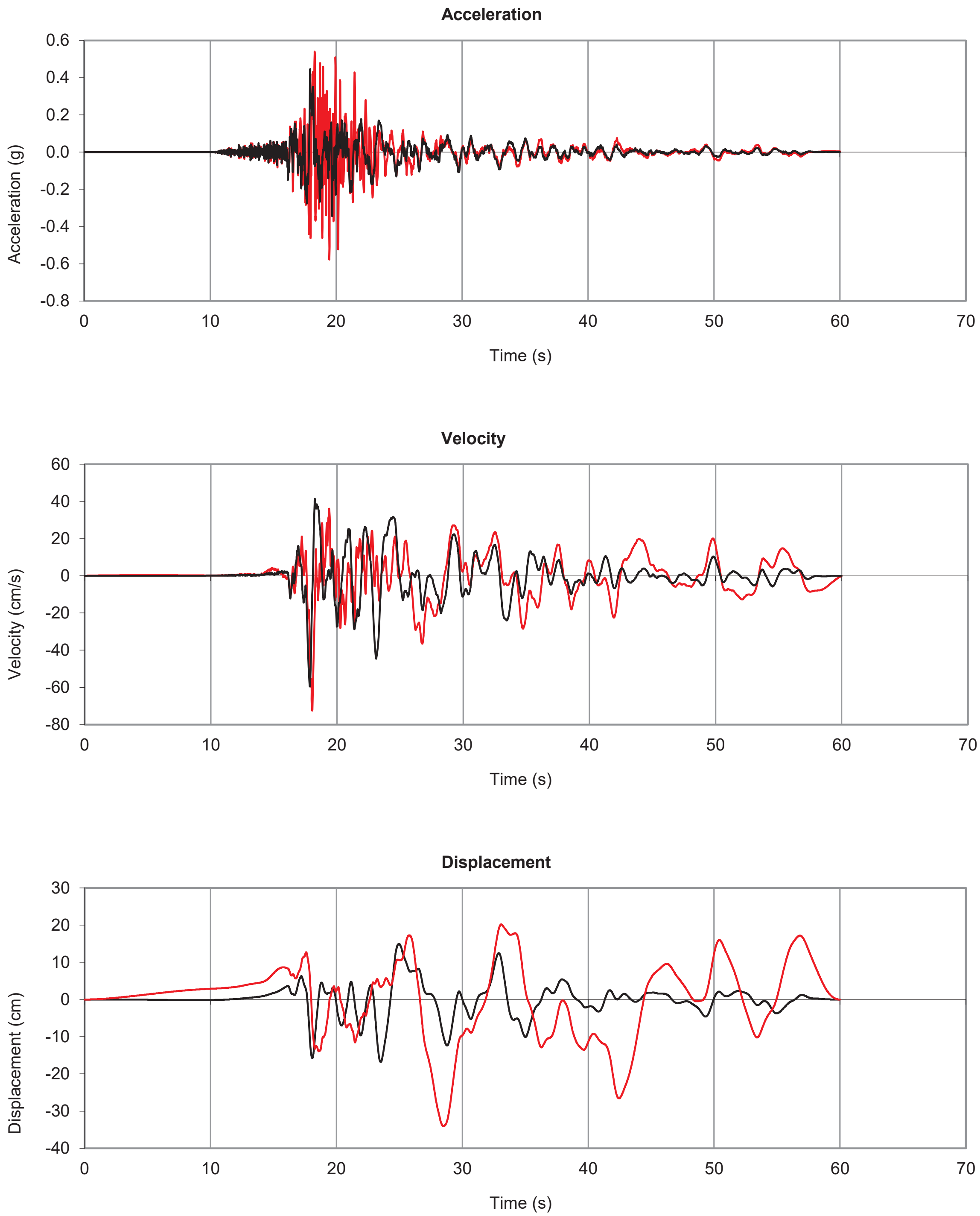
Northridge-01 - RSN1044 - FN (GM-6)
Building X - Redmond, Washington

Figure E-12

Acceleration, Velocity, and Displacement Time Series

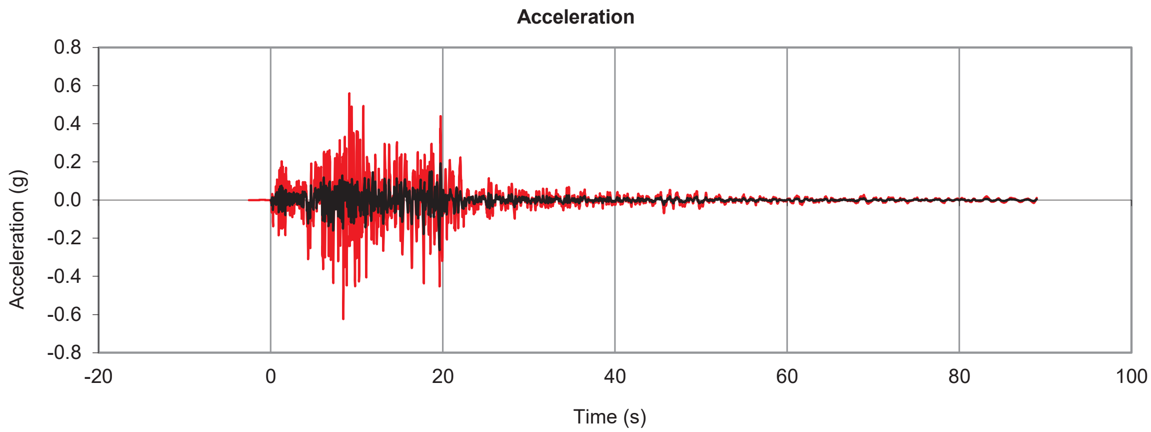


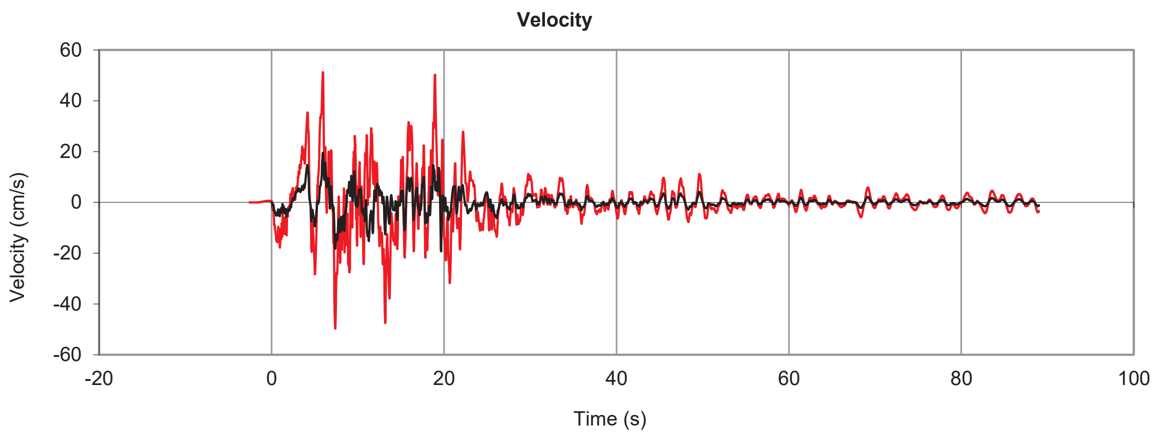
GeoEngineers

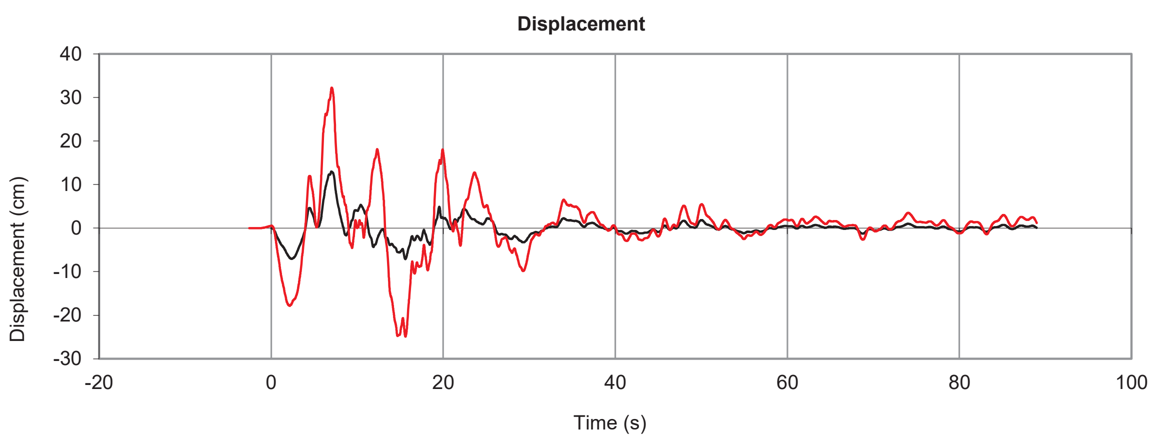
Chuetsu-oki - RSN4847 - FN (GM-7)
Building X - Redmond, Washington

Figure E-13

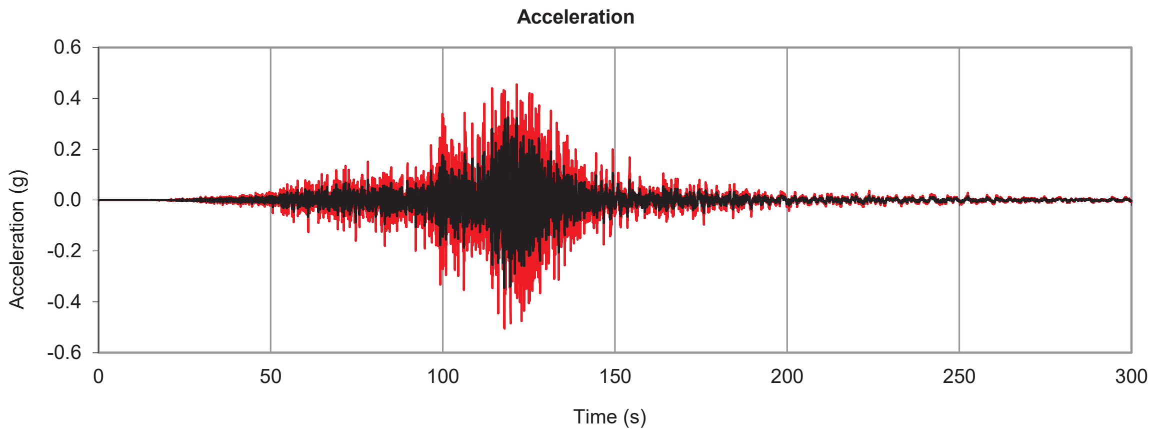
Acceleration, Velocity, and Displacement Time Series

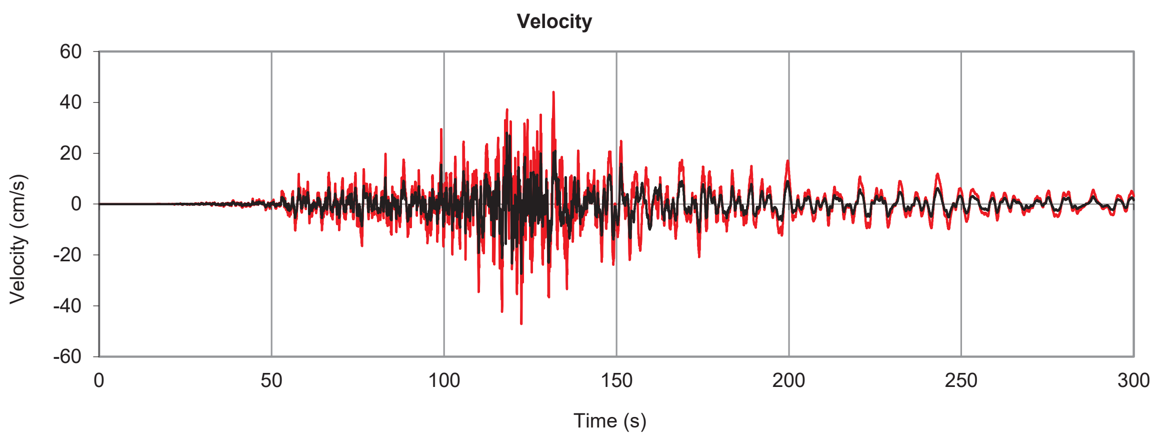


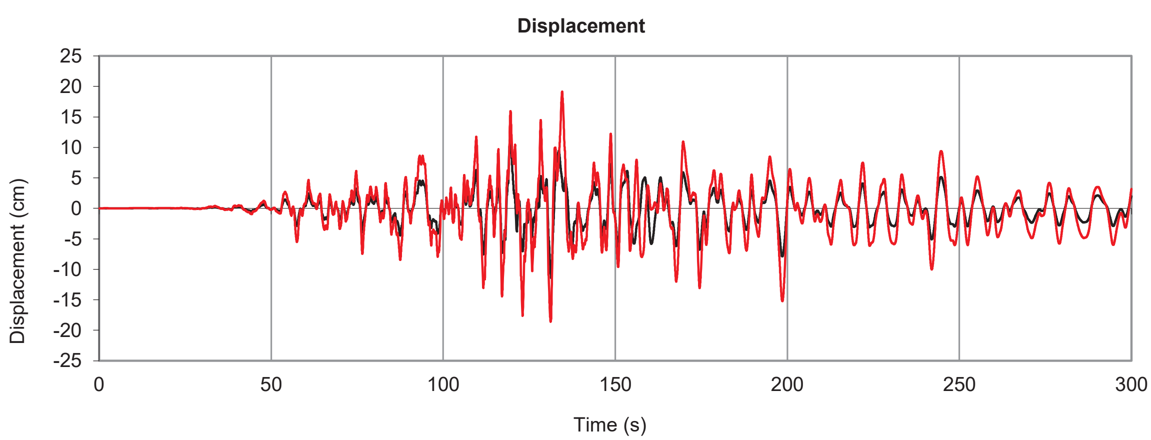




Acceleration, Velocity, and Displacement Time Series







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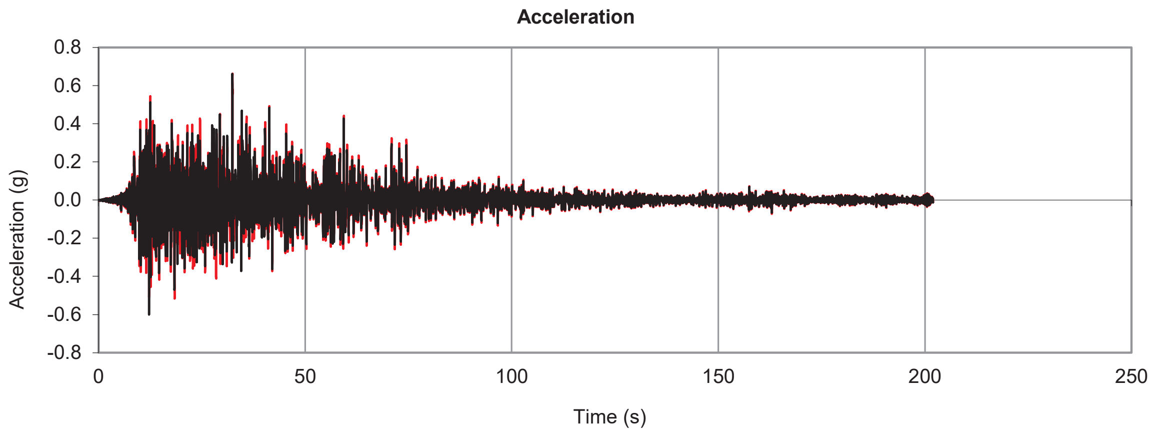
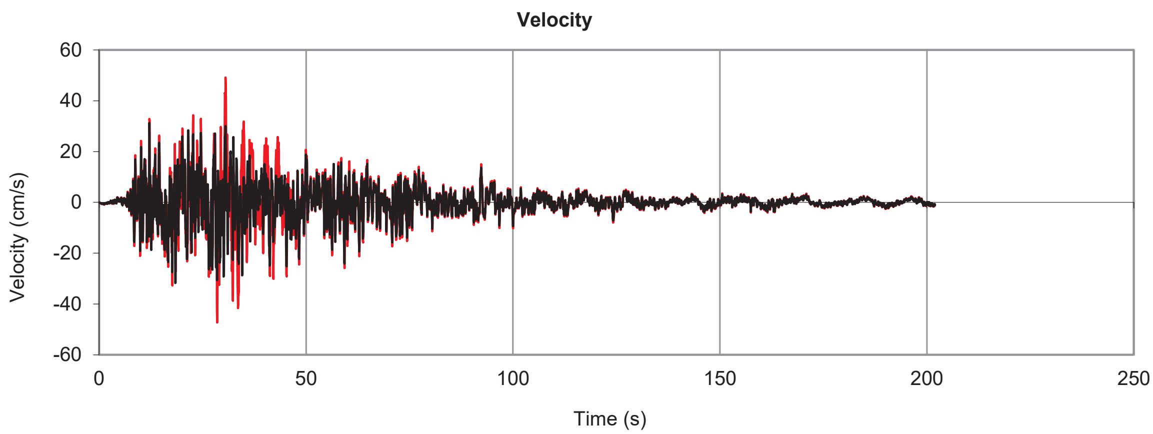
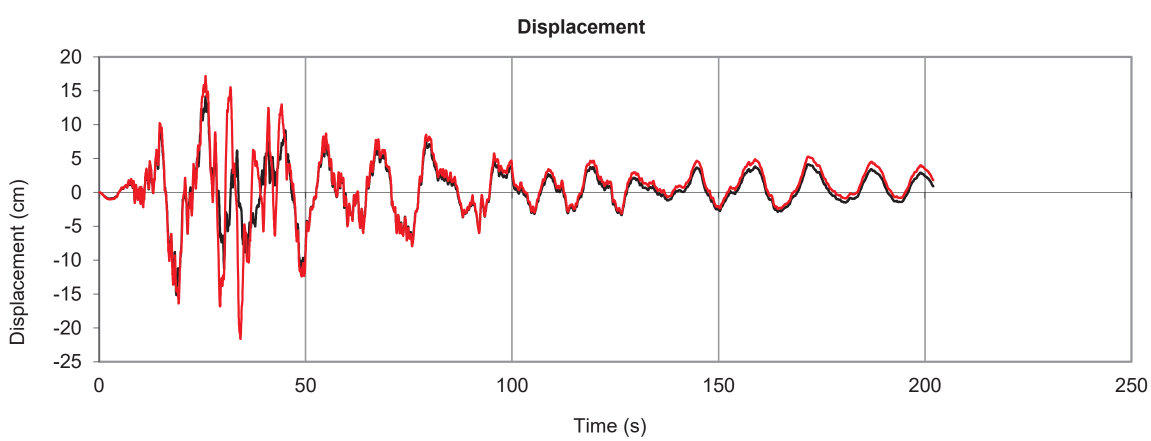
Tohoku - Ujiie TCGH12 - EW (GM-9)
Building X - Redmond, Washington

Figure E-15

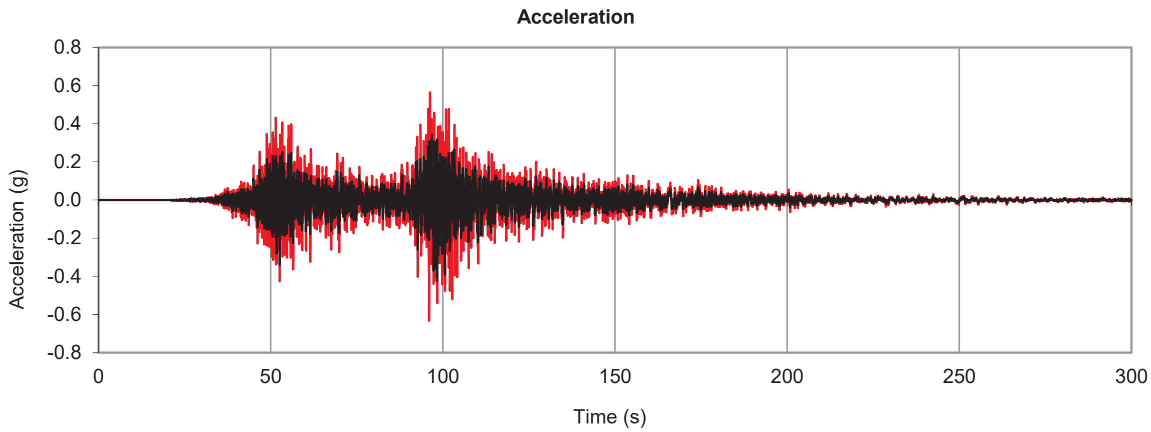
Acceleration, Velocity, and Displacement Time Series

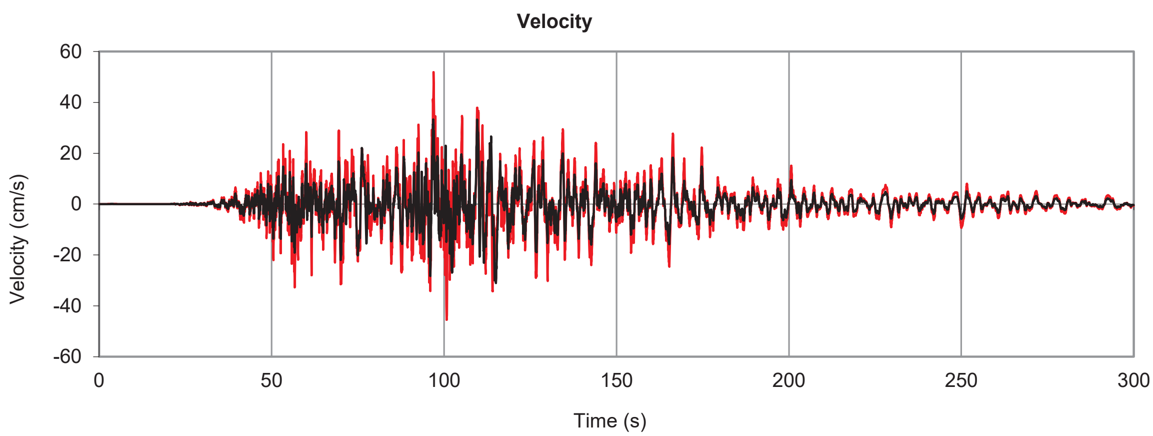


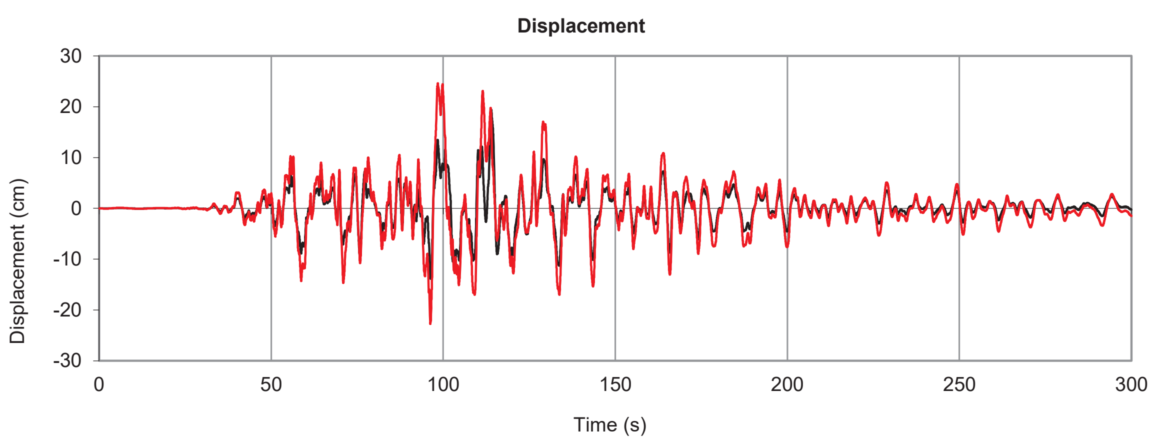




Acceleration, Velocity, and Displacement Time Series

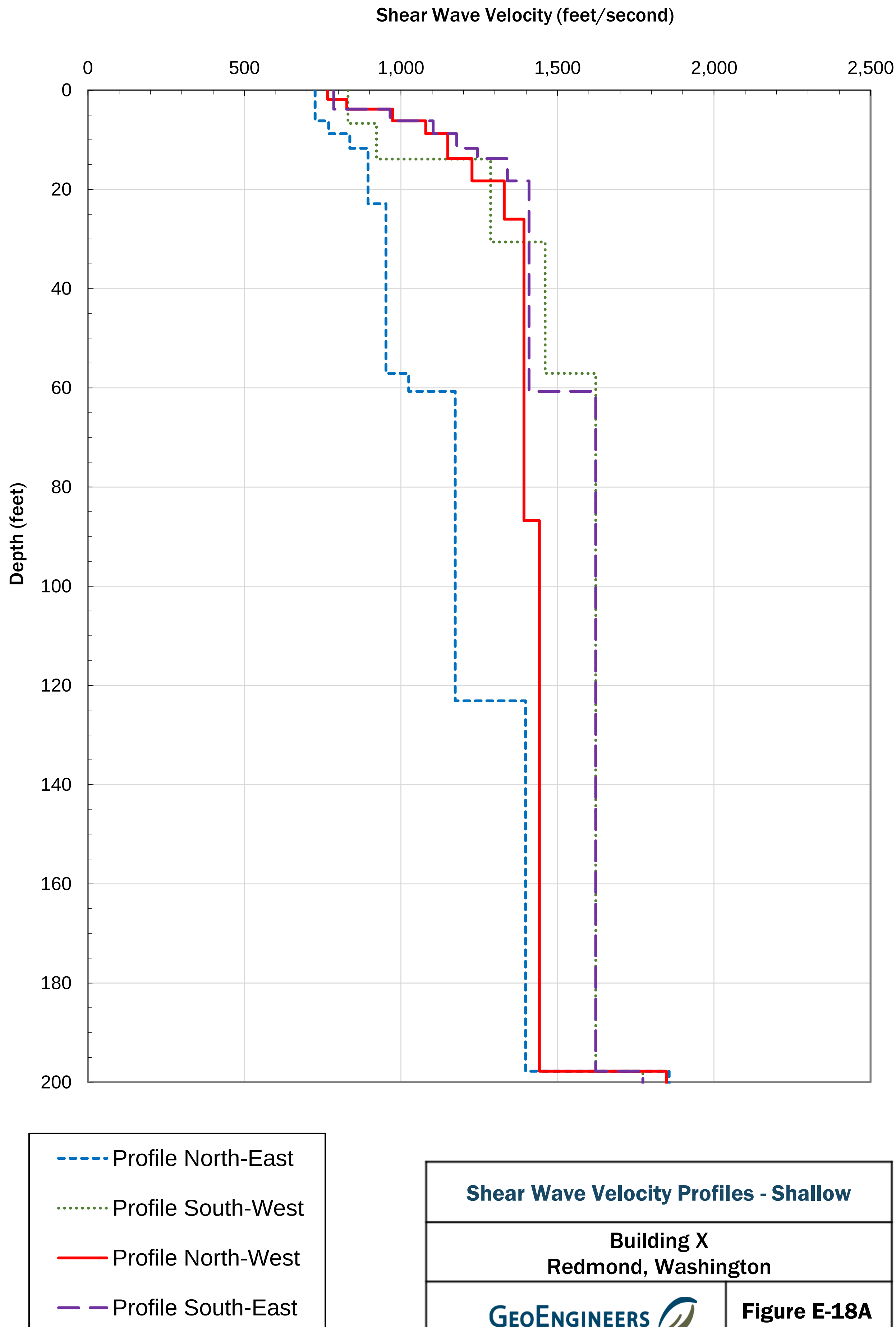






Tohoku - Onoda MYGH05 - EW
Building X - Redmond, Washington

Figure E-17



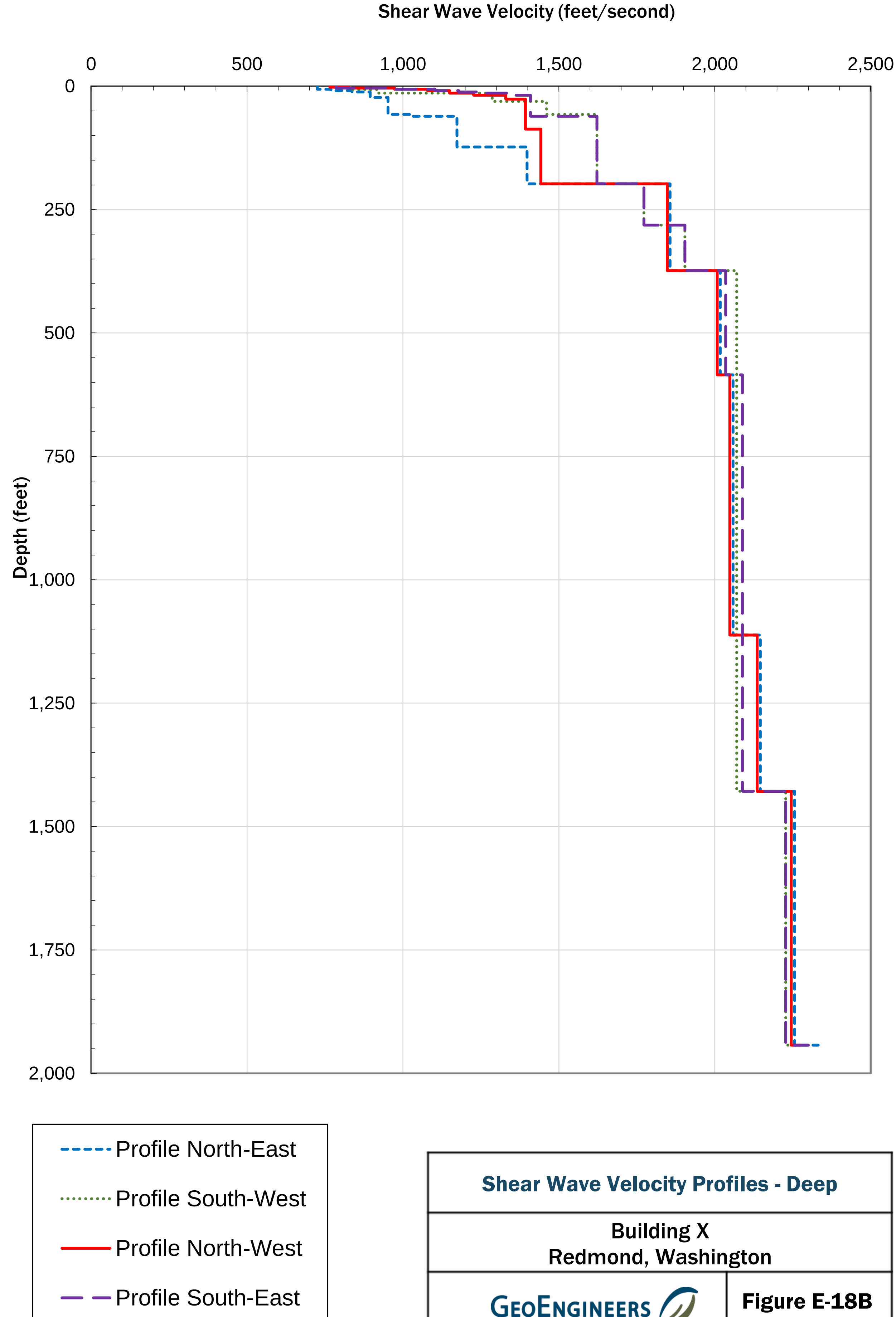
Shear Wave Velocity Profiles - Shallow

Building X
Redmond, Washington

GeoEngineers

Figure E-18A

Project: 23237-002-01 Executed: 11/07/2018



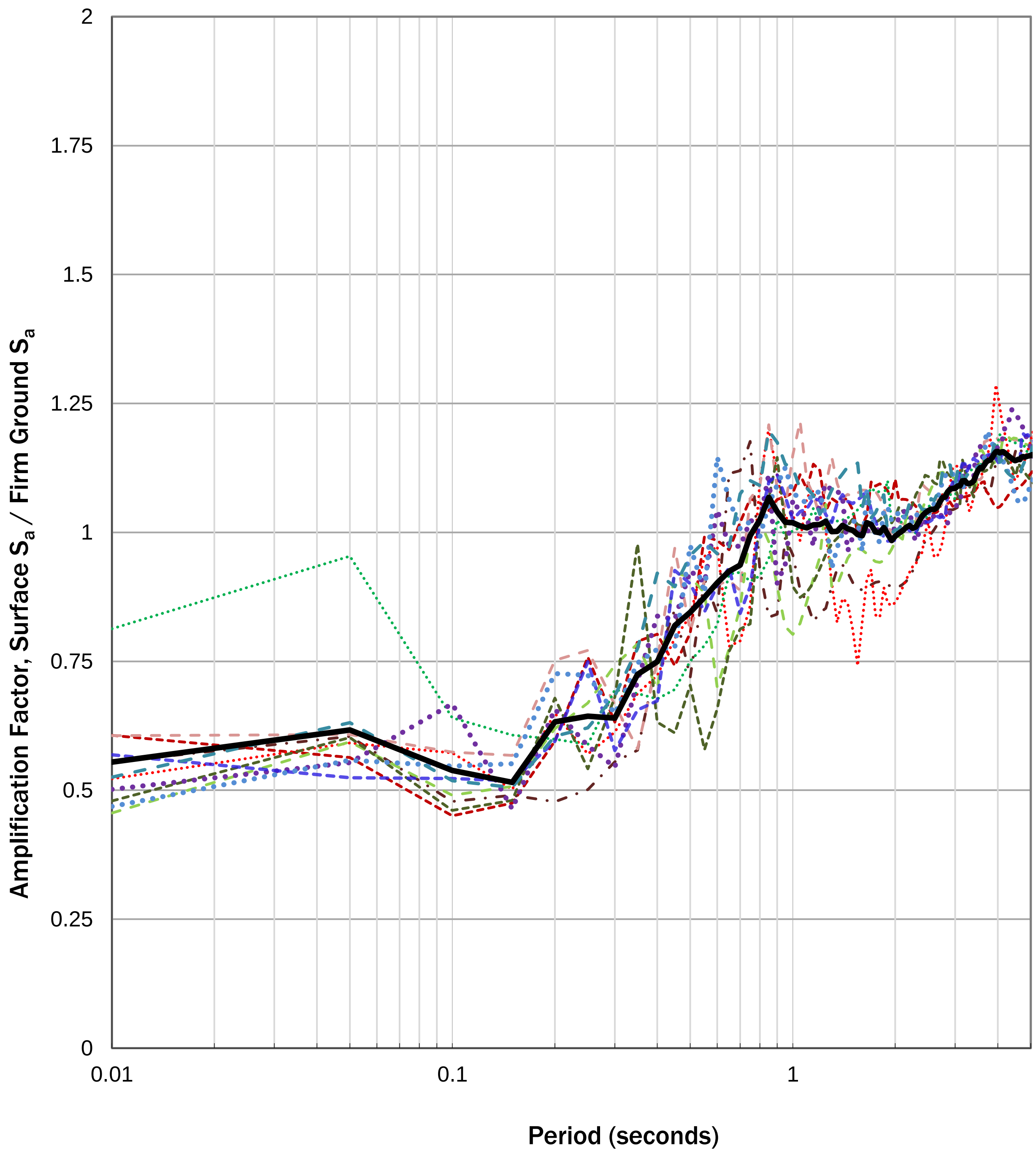
Shear Wave Velocity Profiles - Deep

Building X
Redmond, Washington

GeoEngineers

Figure E-18B

Project: 23237-002-01 Executed: 11/07/2018



Project: 00144-488-00 Executed: 09/07/2018

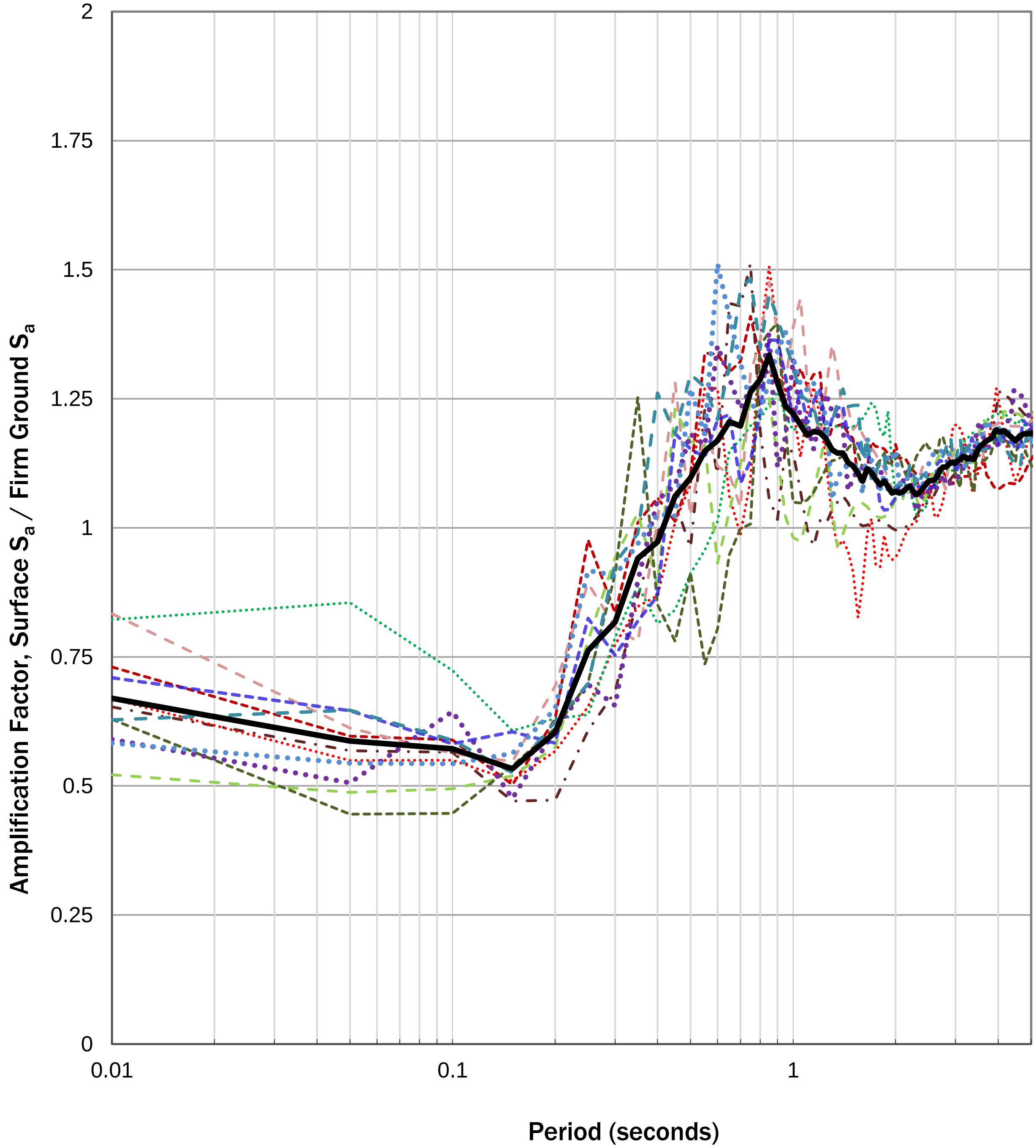
Legend

[C] Crustal Event

[IF] Interface Subduction Zone Event

[IS] Intraslab Subduction Zone Event (Bennioff)

Soil Amplification Factor Profile North-West	
Building X Redmond, Washington	
GeoEngineers	Figure E-19



Legend

[C] Crustal Event

[IF] Interface Subduction Zone Event

[IS] Intraslab Subduction Zone Event (Benniof)

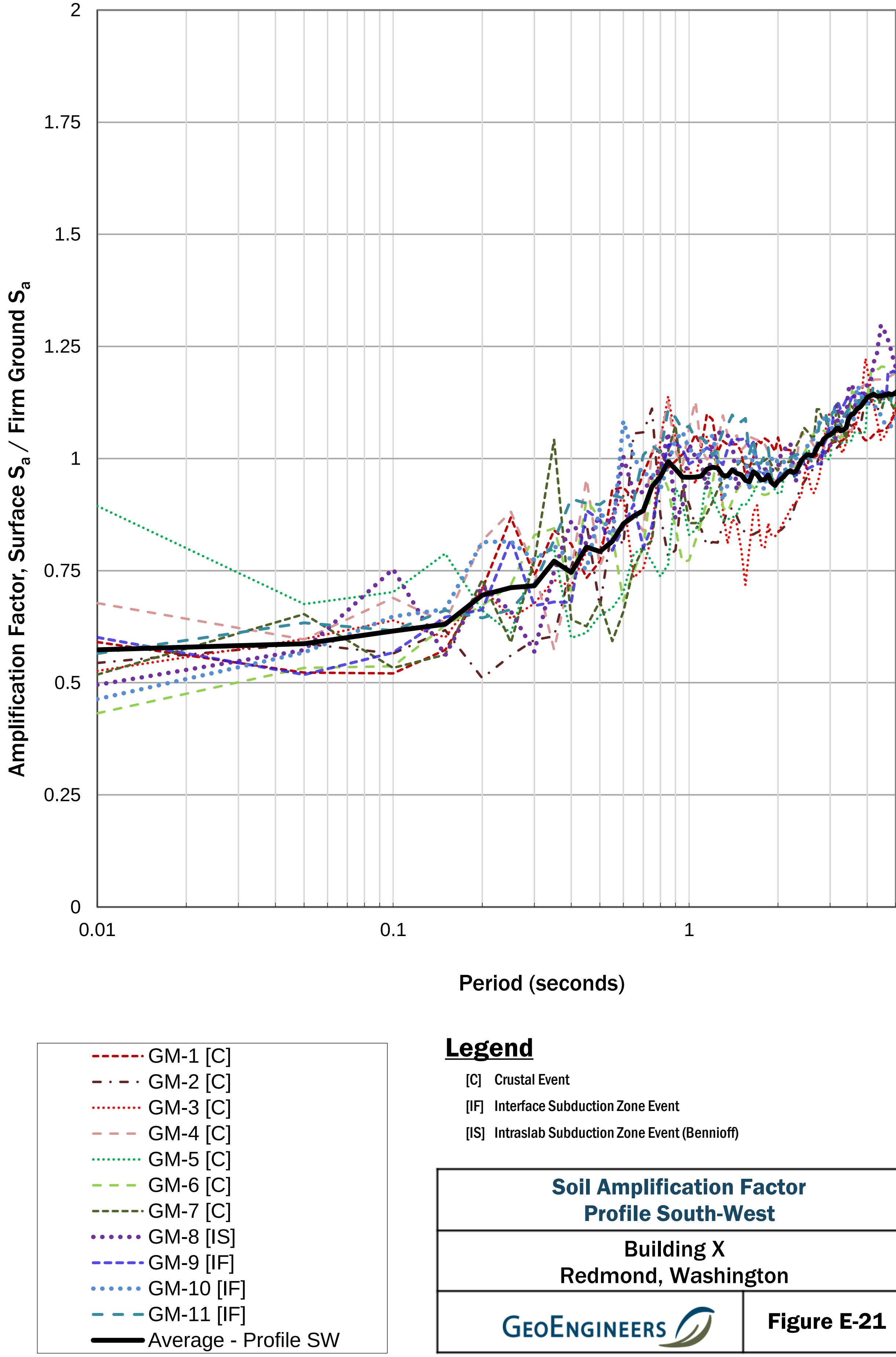
Soil Amplification Factor
Profile North-East

Building X
Redmond, Washington

GeoEngineers

Figure E-20

Project: 00144-488-00 Executed: 09/07/2018



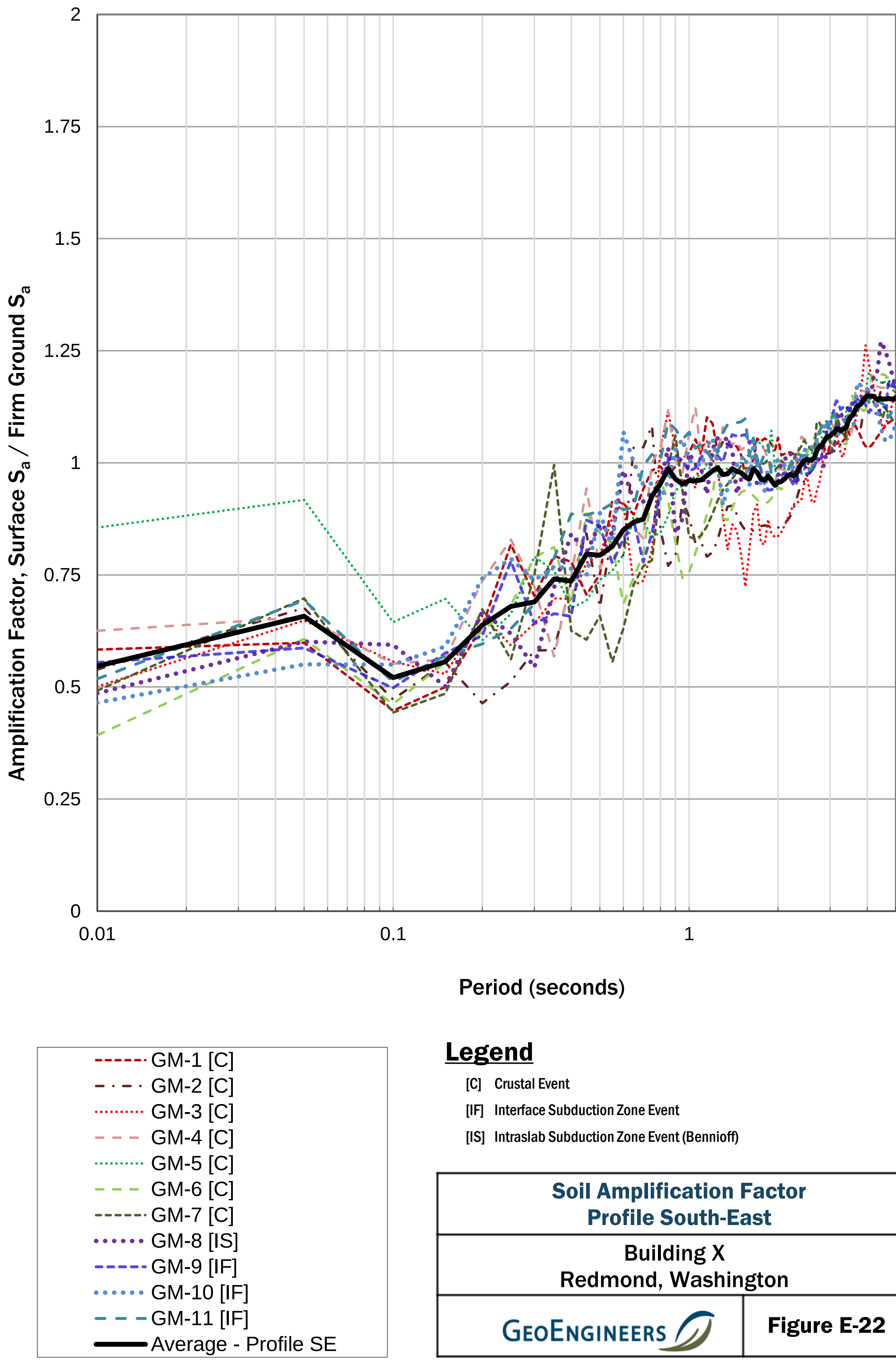
Legend

[C] Crustal Event

[IF] Interface Subduction Zone Event

[IS] Intraslab Subduction Zone Event (Bennioff)

Soil Amplification Factor **Profile South-West**	
Building X **Redmond, Washington**	
GeoEngineers	**Figure E-21**



Legend

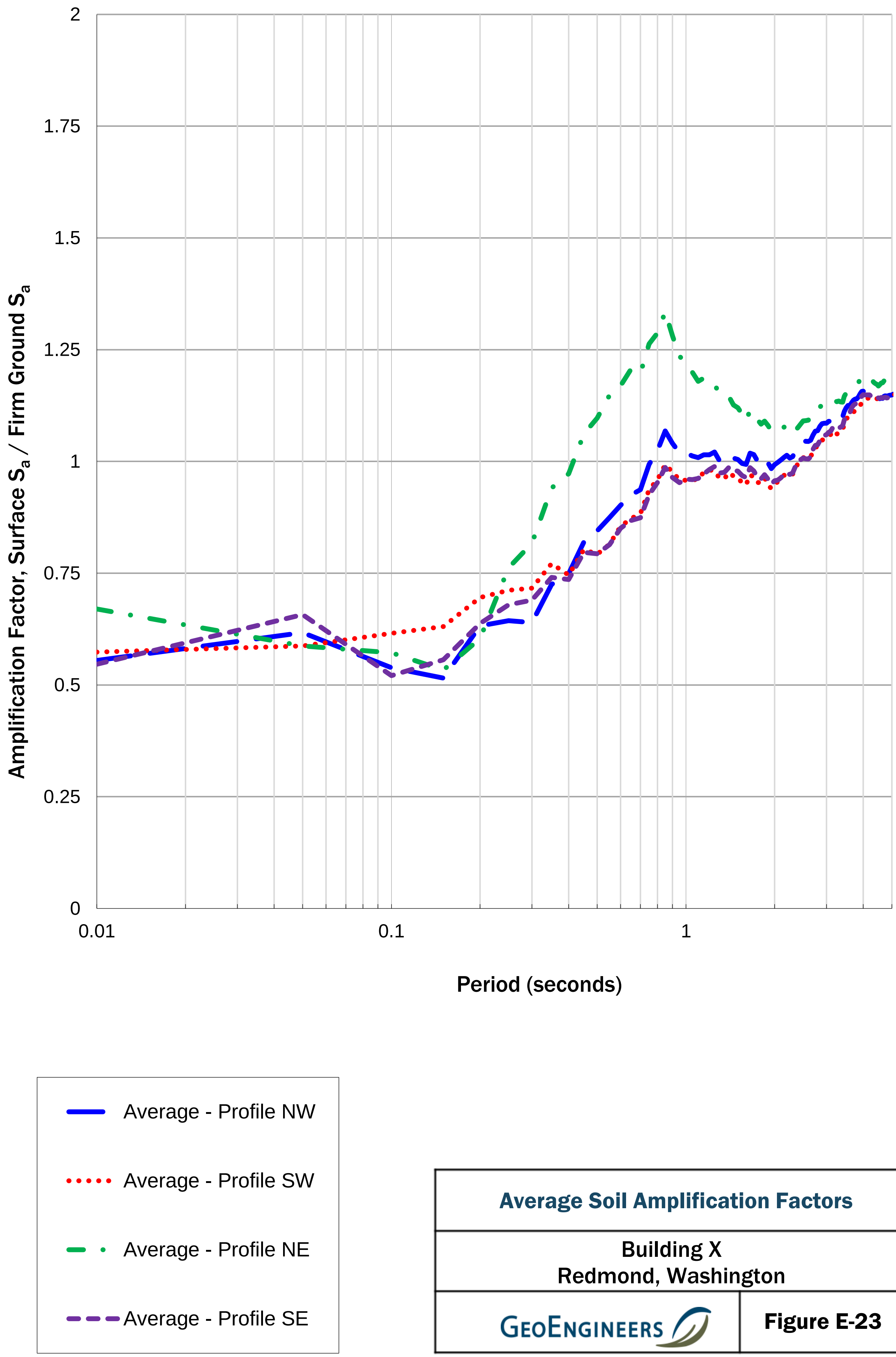
[C] Crustal Event

[IF] Interface Subduction Zone Event

[IS] Intraslab Subduction Zone Event (Bennioff)

Soil Amplification Factor Profile South-East	
Building X Redmond, Washington	
GeoEngineers	Figure E-22

Project: 00144-488-00 Executed: 09/07/2018



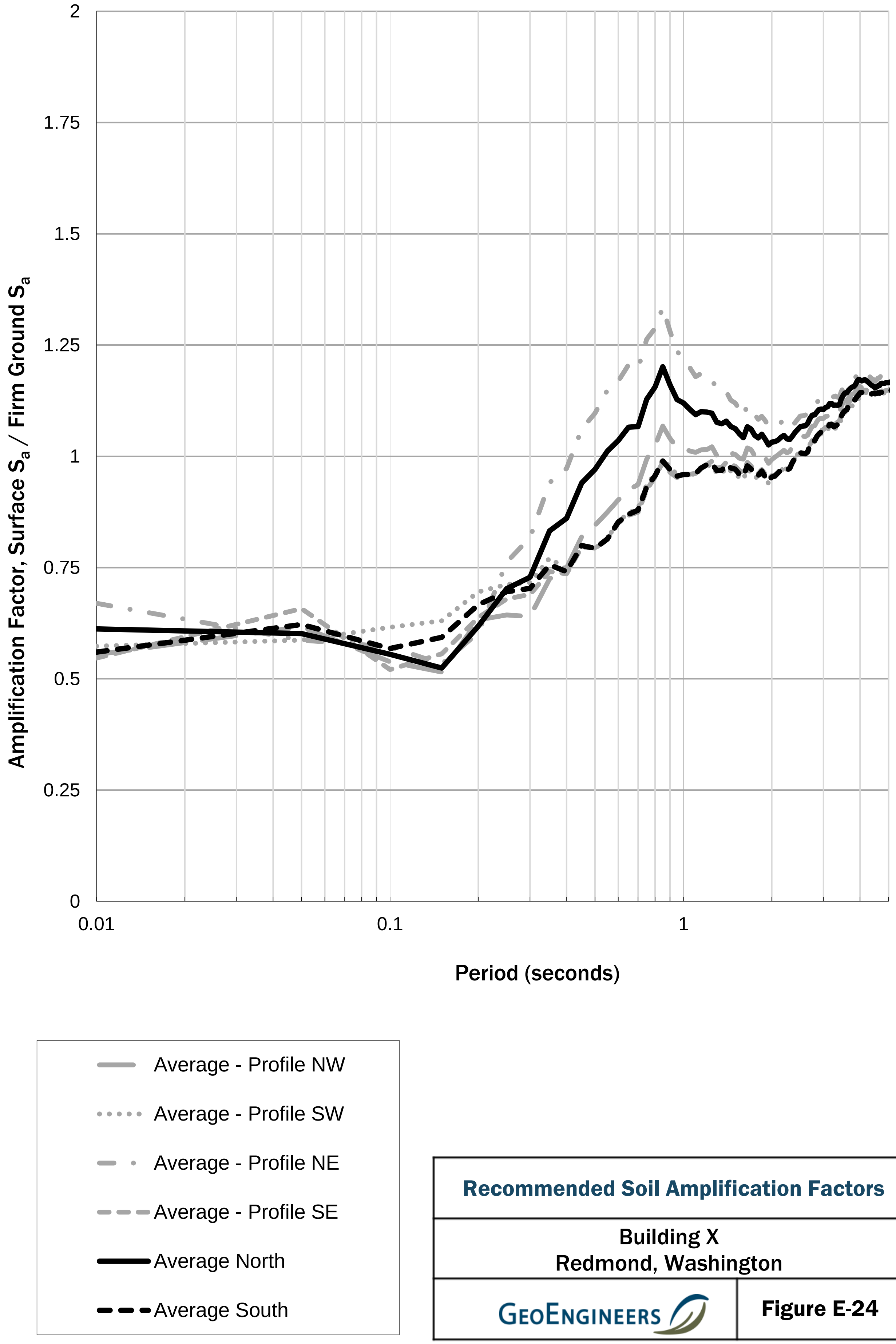
Average Soil Amplification Factors

Building X
Redmond, Washington

GEOENGINEERS

Figure E-23

Project: 00144-488-00 Executed: 09/07/2018



Recommended Soil Amplification Factors

Building X
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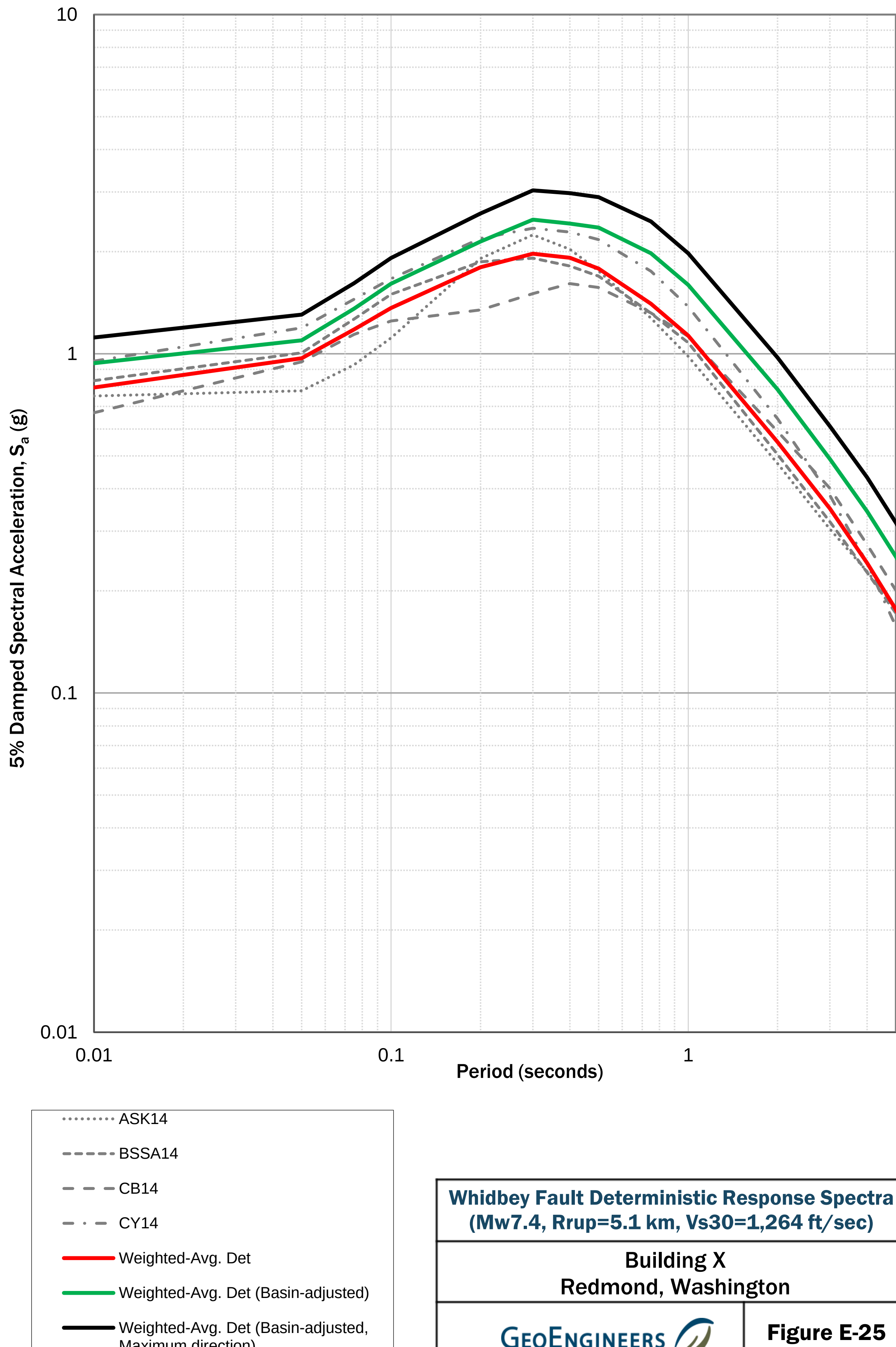
Figure E-24



Whidbey Fault Deterministic Response Spectra (Mw7.4, Rrup=5.1 km, Vs30=1,264 ft/sec)

Building X
Redmond, Washington

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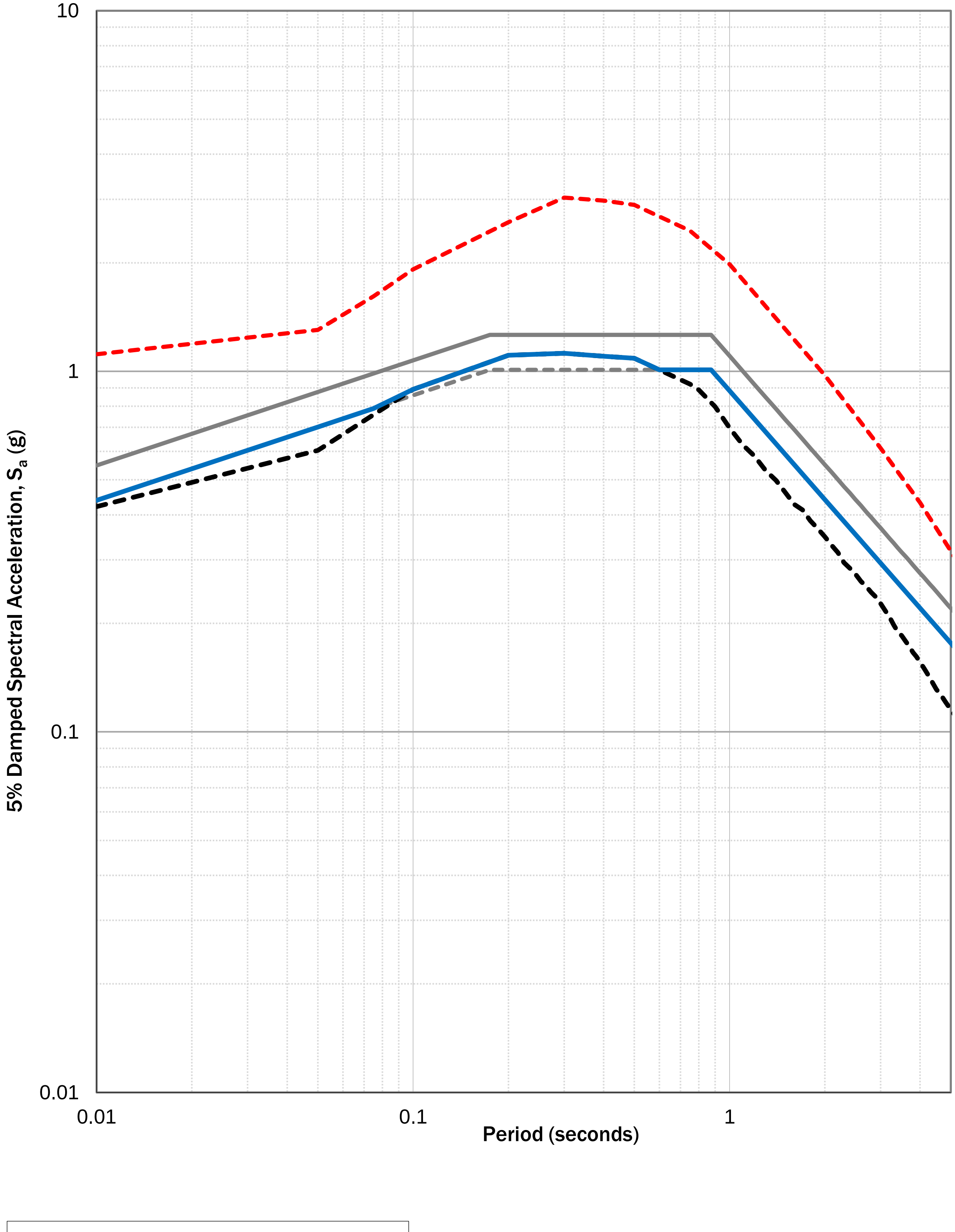
Figure E-25

Project: 23237-002-01 Executed: 11/2/2018



Recommended Site-Specific MCE_R
Response Spectrum - North

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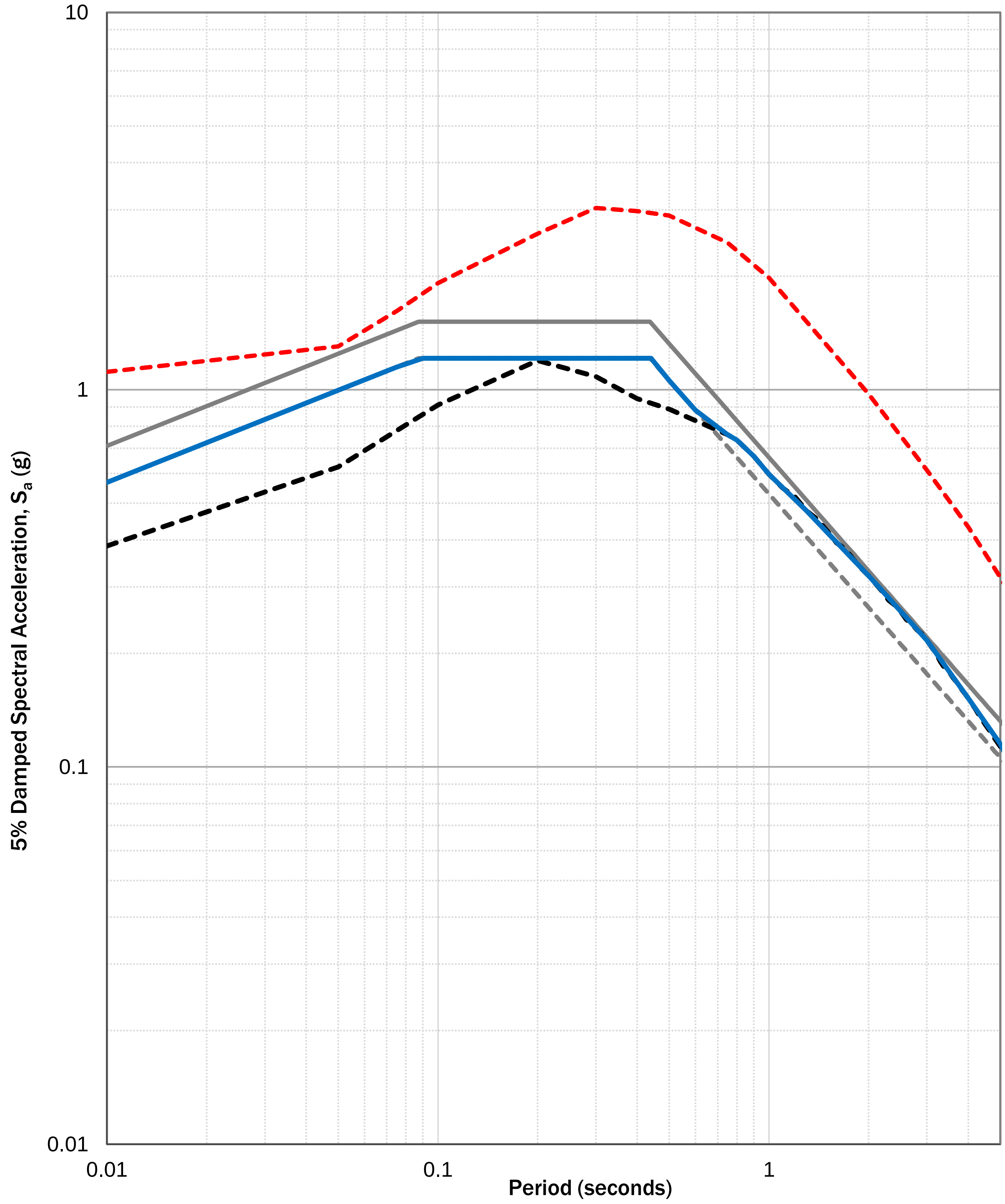
Figure E-26

Project: 23237-002-01 Executed: 11/2/2018



Recommended Site-Specific MCE_R
Response Spectrum - South

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Figure E-27

Project: 23237-002-01 Executed: 11/2/2018

APPENDIX F
Slug Testing

APPENDIX F
SLUG TESTING

A series of slug tests were conducted in three of the monitoring wells at the project site between December 12 and 13, 2018 as a basis for estimating hydraulic conductivity in the aquifer zones encountered at the Building X site.

Two slug test cycles, each comprised of one falling-head and one rising-head test phase, were performed in each of the tested wells.

Methodology

Prior to testing each well, a vented 0-15 or 0-30 psig INW PT2X pressure transducer was installed in the well and set to record water levels every two seconds throughout the testing period. A slug (weighted 5-foot length of sealed polyvinyl chloride [PVC] casing) of known volume was rapidly lowered into the well to instantaneously displace the water column upward and force water to flow out of the well into the surrounding aquifer zone. The declining water level in the well immediately after slug insertion was monitored until it returned to the approximate initial water level (comprising the falling-head stage of the test).

The slug was then rapidly removed, instantaneously lowering the water column and forcing water to flow back into the well from the surrounding aquifer zone. The rising water level was monitored until it returned to the approximate initial water level (comprising the rising-head stage of the test). Groundwater levels were measured with a pressure sensor (with built-in data logger) and manual electronic water level meter before, during and after each aquifer slug test.

Slug Test Analysis

The Bouwer & Rice (1976) method for unconfined aquifers (as presented in Dawson and Istok 1991) was used to calculate the hydraulic conductivity. Graphs showing the data from each test in the form of normalized head (log scale) against elapsed time are presented in Figures F-1 through F-3. Normalized head is defined as the measured displacement of water (in feet), divided by the initial or instantaneous displacement created at the beginning of each slug test.

The average hydraulic conductivity for the tested zone is calculated from the slope of a straight line fitted through the normalized head data, with emphasis placed on the larger values of normalized head. The resulting values for hydraulic conductivity determined from each test are listed in Table F-1 and span a relatively narrow range.

TABLE F-1. SLUG TEST RESULTS

Well	Type of Slug Test	Hydraulic Conductivity	
		K (ft/day)	K (cm/sec)
Shallow Unconfined Aquifer			
GEI-5	Falling-Head 1	0.06	2.2E-05
	Rising-Head 1	NA[2]	NA
	Average [1]	0.06	2.2E-05
Perched Aquifer Zones			
AMW-04	Falling-Head 1	0.35	1.2E-04
	Rising-Head 1	NA[3]	NA
	Average [1]	0.35	1.2E-04
Deep Confined Aquifer			
GEI-1	Falling-Head 1	0.03	1.2E-05
	Rising-Head 1	0.03	9.9E-06
	Average [1]	0.03	1.1E-05

Notes:

[1] Average values of hydraulic conductivity are calculated as geometric means (not arithmetic means)

[2] Rising-head test not completed due to slug/transducer interference

[3] Test result not reported due to possible well siltation or bioufouling

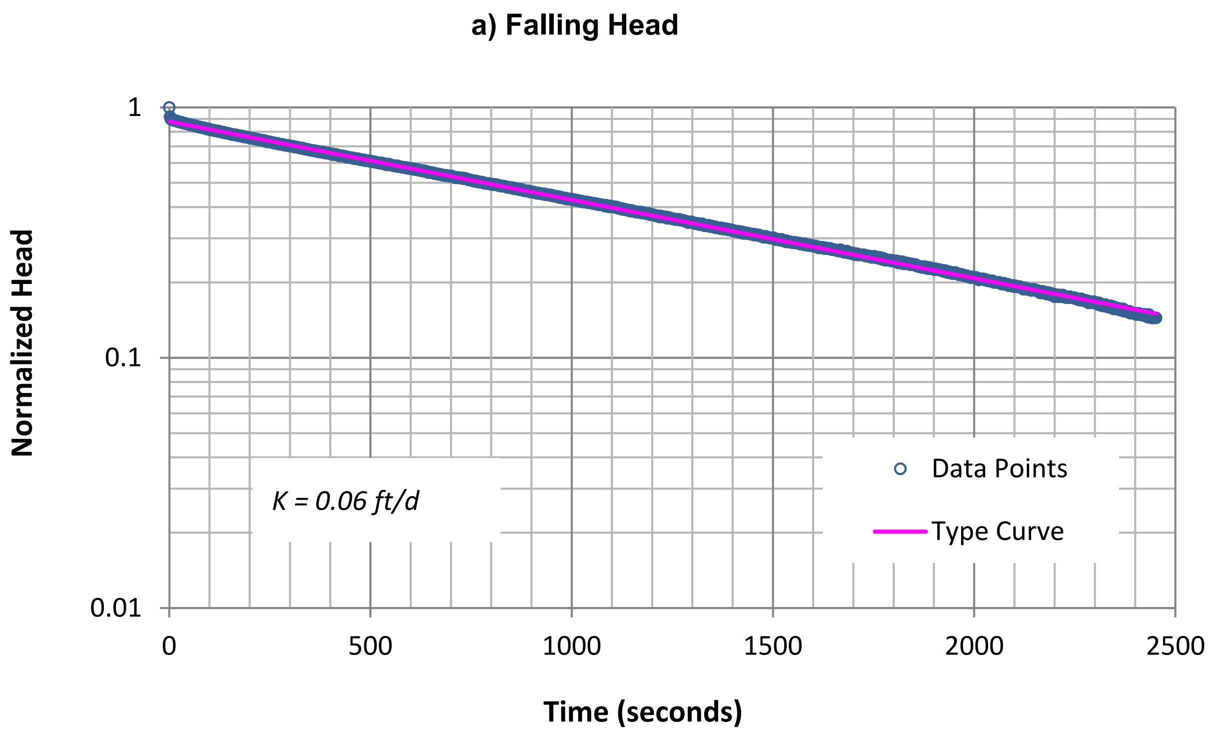
ft/day = feet per day; cm/sec = centimeters per second; NA = not applicable

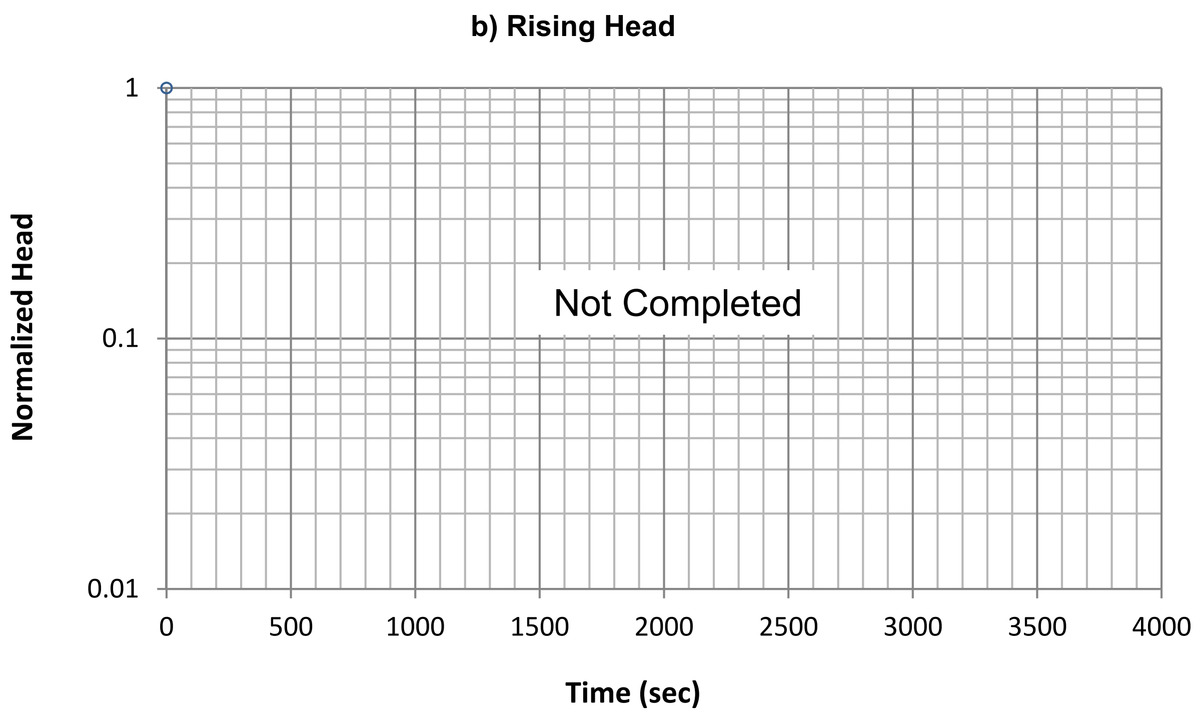
The geometric mean of the values is considered to provide the best estimate of the average hydraulic conductivity for the aquifer zone encountered at GEI-5. With an assumed thickness of 10 feet, the shallow unconfined aquifer zone has a transmissivity of 0.6 square feet per day (ft^2/d).

References

Bouwer, H. and R. C. Rice, 1976. A Slug Test for Determining Hydraulic Conductivity of Unconfined Aquifers with Completely or Partially Penetrating Wells. Water Resources Research, Vol. 12, No. 3, pp. 423-428.

Dawson, K.J. and J.D. Istok, 1991. Aquifer Testing: Design and Analysis of Pumping and Slug Tests. Lewis Publishers, Inc., Chelsea, Michigan.



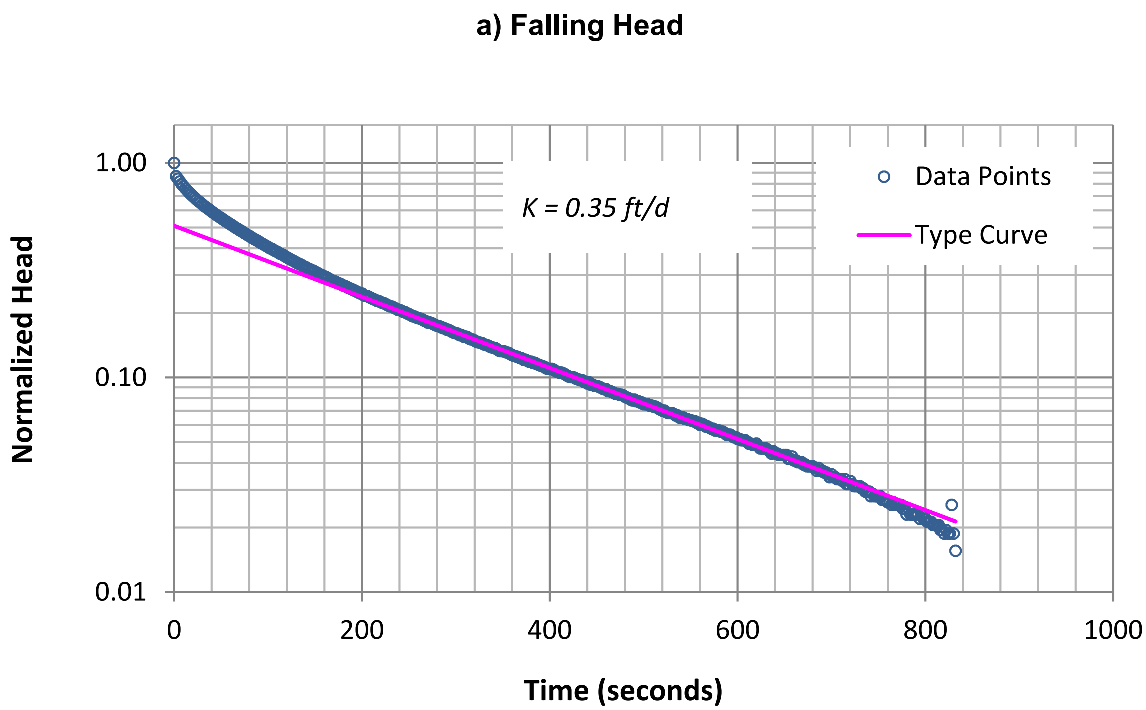


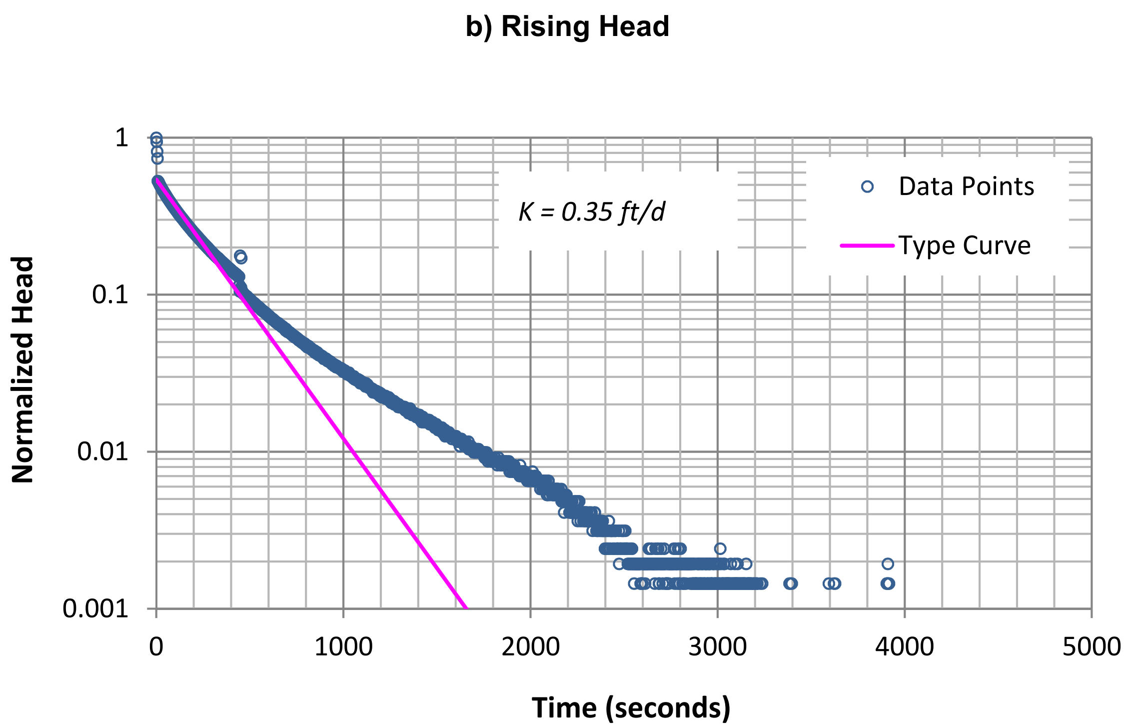
23237-002-01 Date Exported: 02/05/2019

Notes:

1. Water levels were recorded with a INW vented PT2X pressure transducer installed at the bottom of the well and set to record four times per second
2. GEI-5-1 is screened from 15 to 25 feet below ground surface
3. Static water level is 14.59 feet below ground surface on 12/13/2018
4. The slug test was performed by inserting then removing a 1.5 inch diameter PVC slug
5. Aquifer is unconfined from ground surface to a silt layer 23 feet below ground surface
6. Rising head test not completed due to transducer/slug interference

Slug Test Analysis: GEI-5	
Building X Redmond, Washington	
GEOENGINEERS	**Figure F-1**





Notes:

1. Water levels were recorded with a INW vented PT2X pressure transducer installed at the bottom of the well and set to record four times per second
2. AMW094 is screened from 12 to 27 feet below ground surface
3. Static water level is 6.4 feet below ground surface on 12/12/2018
4. The slug test was performed by inserting then removing a 1.5 inch diameter PVC slug
5. Aquifer is unconfined from ground surface to a silt layer 20 feet below ground surface – well is screened within silt layer

23237-002-01 Date Exported: 02/05/2019

Slug Test Analysis: AMW-04

Building X
Redmond, Washington

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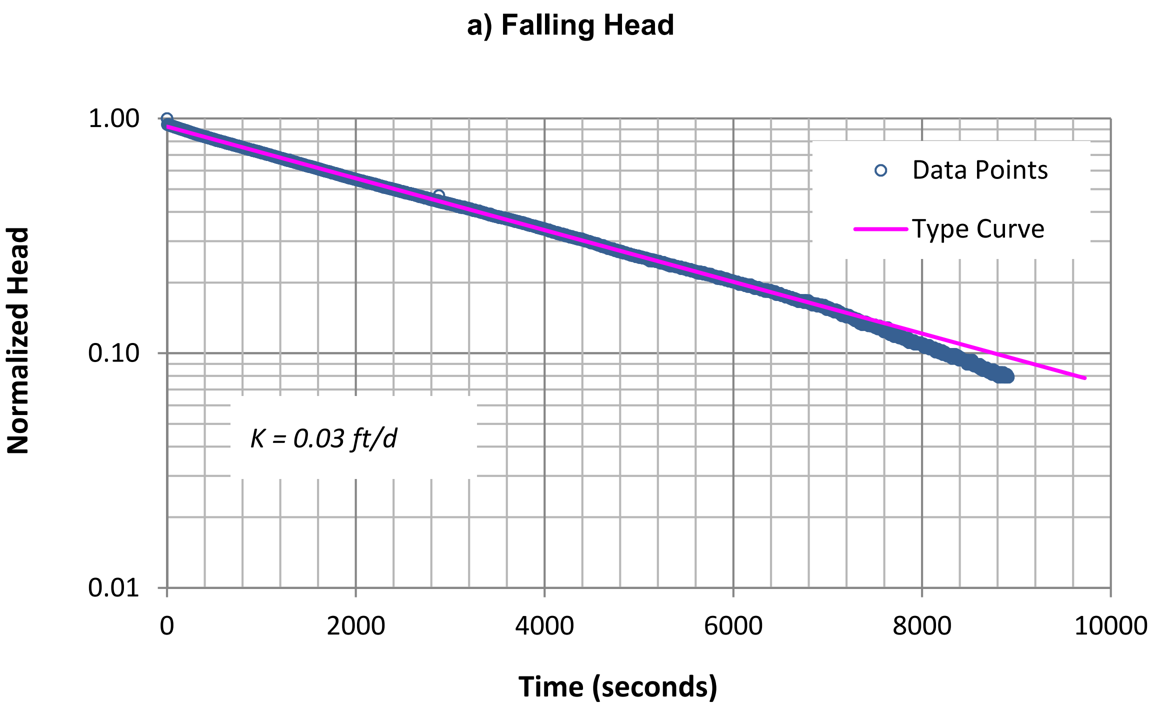
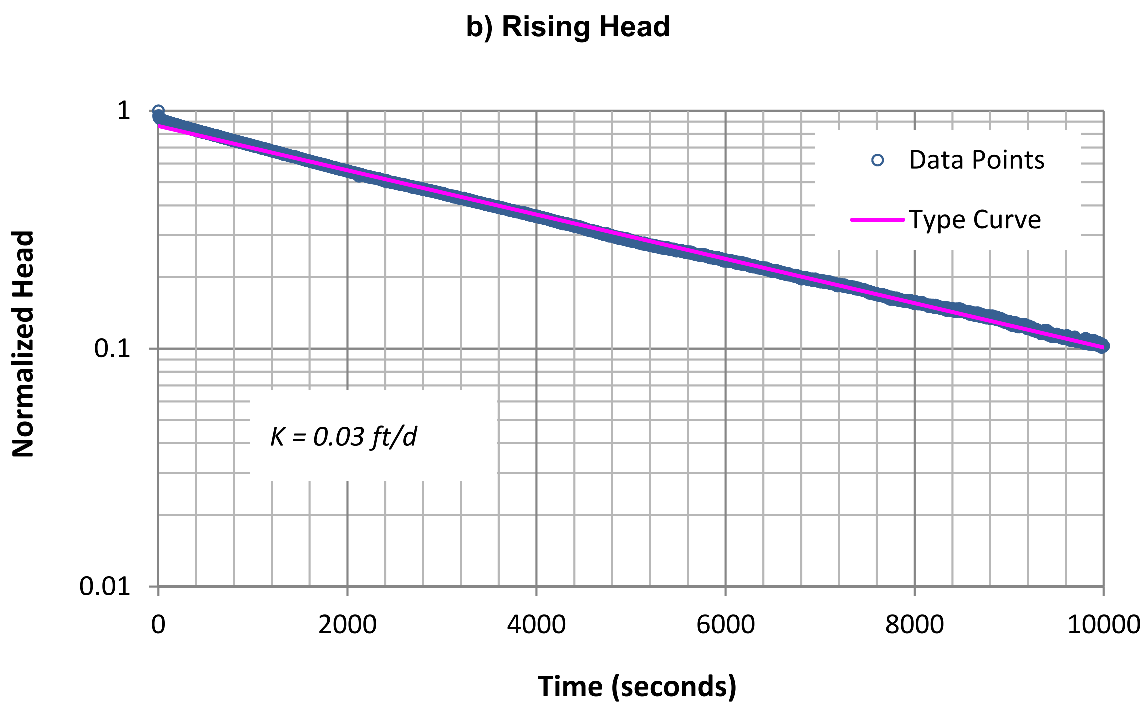
Figure F-2





Notes:

1. Water levels were recorded with a INW vented PT2X pressure transducer installed at the bottom of the well and set to record four times per second
2. GEI-1 is screened from 61.75 to 71.75 feet below ground surface
3. Static water level is 7.9 feet below ground surface on 12/12/2018
4. The slug test was performed by inserting then removing a 1.5 inch diameter PVC slug
5. Aquifer is confined from ground surface to a sandy layer 63 feet below ground surface

Slug Test Analysis: GEI-1

Building X
Redmond, Washington

GeoEngineers

Figure F-3

APPENDIX G
Ground Anchor Load Tests and Shoring Monitoring Program

APPENDIX G
GROUND ANCHOR LOAD TESTS AND SHORING MONITORING PROGRAM

Ground Anchor Load Testing

The locations of the load tests shall be approved by the Engineer and shall be representative of the field conditions. Load tests shall not be performed until the nail/tieback grout and shotcrete wall facing, where present, have attained at least 50 percent of the specified 28-day compressive strengths.

Where temporary casing of the unbonded length of test nails/tiebacks is provided, the casing shall be installed to prevent interaction between the bonded length of the nail/tieback and the casing/testing apparatus.

The testing equipment shall include two dial gauges accurate to 0.001 inch, a dial gauge support, a calibrated jack and pressure gauge, a pump and the load test reaction frame. The dial gauge should be aligned within 5 degrees of the longitudinal nail/tieback axis and shall be supported independently from the load frame/jack and the shoring wall. The hydraulic jack, pressure gauge and pump shall be used to apply and measure the test loads.

The jack and pressure gauge shall be calibrated by an independent testing laboratory as a unit. The pressure gauge shall be graduated in 100 pounds per square inch (psi) increments or less and shall have a range not exceeding twice the anticipated maximum pressure during testing unless approved by the Engineer. The ram travel of the jack shall be sufficient to enable the test to be performed without repositioning the jack.

The jack shall be supported independently and centered over the nail/tieback so that the nail/tieback does not carry the weight of the jack. The jack, bearing plates and stressing anchorage shall be aligned with the nail/tieback. The initial position of the jack shall be such that repositioning of the jack is not necessary during the load test.

The reaction frame should be designed/sized such that excessive deflection of the test apparatus does not occur and that the testing apparatus does not need to be repositioned during the load test. If the reaction frame bears directly on the shoring wall facing, the reaction frame should be designed so as not to damage the facing.

Verification Tests

Prior to production soil nail/tieback installation, at least two soil nails/tiebacks for each soil type shall be tested to validate the design pullout value. All test nails/tiebacks shall be installed by the same methods, personnel, material and equipment as the production anchors. Changes in methods, personnel, material or equipment may require additional verification testing as determined by the Engineer. At least two successful verification tests shall be performed for each installation method and each soil type. The nails/tiebacks used for the verification tests may be used as production nails/tiebacks if approved by the Engineer.

For soil nails, the unbonded length of the test nails shall be at least 3 feet unless approved otherwise by the Engineer. The bond length of the test nails shall not be less than 10 feet and shall not be longer than the bond length that would prevent testing to 200 percent of the design load while not exceeding the allowable bar load. The allowable bar load during testing shall not exceed 80 percent of the steel ultimate strength for Grade 150 bars or 90 percent of the steel ultimate strength for Grade 60 and 75 bars. The allowable tieback load should not exceed 80 percent of the steel ultimate strength.

For soil nails, the design test load shall be determined by multiplying the bond length of the nail times the design load pullout resistance (load transfer). Tieback design test loads should be the design load specified on the shoring drawings. Verification test nails/tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL	1 minute
1.5DL	60 minutes
1.75DL	1 minute
2.0DL	10 minutes

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied. Nail/tieback deflections during the 1.5DL test load shall be recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50 and 60 minutes.

Proof Tests

Proof tests shall be completed on approximately 5 percent of the production nails at locations selected by the owner's representative. Additional testing may be required where nail installation methods are substandard. Proof tests shall be completed on each production tieback.

For soil nails, the unbonded length of the test nails shall be at least 3 feet unless approved otherwise by the Engineer. The bond length of the test nails shall not be less than 10 feet and shall not be longer than the bond length that would prevent testing to 200 percent of the design load while not exceeding the allowable bar load. The allowable bar load during testing shall not exceed 80 percent of the steel ultimate strength for Grade 150 bars or 90 percent of the steel ultimate strength for Grade 60 and 75 bars. The allowable tieback load should not exceed 80 percent of the steel ultimate strength.

For soil nails, the design test load shall be determined by multiplying the bond length of the nail times the design load pullout resistance (load transfer). Tieback design test loads should be the design load specified on the shoring drawings. Proof test nails/tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL (soil nails)	1 minute
1.33DL (tiebacks)	10 minutes
1.5DL (soil nails)	

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied. Nail/tieback deflections during the 1.33DL and 1.5DL test loads shall be recorded at 1, 2, 3, 5, 6 and 10 minutes.

Depending upon the nail/tieback deflection performance, the load hold period at 1.33DL (tiebacks) or 1.5DL (soil nails) may be increased to 60 minutes. Nail/tieback movement shall be recorded at 1, 2, 3, 5, 6 and 10 minutes. If the nail/tieback deflection between 1 and 10 minutes is greater than 0.04 inches, the 1.33DL/1.5DL load shall be continued to be held for a total of 60 minutes and deflections recorded at 20, 30, 50 and 60 minutes.

Test Nail/Tieback Acceptance

A test nail/tieback shall be considered acceptable when:

1. For verification tests, a nail/tieback is considered acceptable if the creep rate is less than 0.08 inches per log cycle of time between 6 and 60 minutes and the creep rate is linear or decreasing throughout the creep test load hold period;
2. For proof tests, a nail/tieback is considered acceptable if the creep rate is less than 0.04 inches per log cycle of time between 1 and 10 minutes or the creep rate is less than 0.08 inches per log cycle of time between 6 and 60 minutes, and the creep rate is linear or decreasing throughout the creep test load hold period;
3. The total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length; and
4. Pullout failure does not occur. Pullout failure is defined as the load at which continued attempts to increase the test load result in continued pullout of the test nail/tieback.

Acceptable proof-test nails/tiebacks may be incorporated as production nails/tiebacks provided that the unbonded test length of the nail/tieback hole has not collapsed and the test nail/tieback length and bar size/number of strands are equal to or greater than the scheduled production nail/tieback at the test location. Test nails/tiebacks meeting these criteria shall be completed by grouting the unbonded length. Maintenance of the temporary unbonded length for subsequent grouting is the contractor's responsibility.

The Engineer shall evaluate the verification test results. Nail/tieback installation techniques that do not satisfy the nail/tieback testing requirements shall be considered inadequate. In this case, the contractor shall propose alternative methods and install replacement verification test nails/tiebacks.

The Engineer may require that the contractor replace or install additional production nails/tiebacks in areas represented by inadequate proof tests.

Shoring Monitoring

Preconstruction Survey

A shoring monitoring program should be established to monitor the performance of the temporary shoring walls and to provide early detection of deflections that could potentially damage nearby improvements. We recommend that a preconstruction survey of adjacent improvements, such as streets, utilities and buildings, be performed prior to commencing construction. The preconstruction survey should include a video or photographic survey of the condition of existing improvements to establish the preconstruction condition, with special attention to existing cracks in streets or buildings.

Optical Survey

The shoring monitoring program should include an optical survey monitoring program. The recommended frequency of monitoring should vary as a function of the stage of construction as presented in the following table.

Construction Stage	Monitoring Frequency
During excavation and until wall movements have stabilized	Twice weekly
During excavation if lateral wall movements exceed 1 inch and until wall movements have stabilized	Three times per week
After excavation is complete and wall movements have stabilized, and before the floors of the building reach the top of the excavation	Twice monthly

Monitoring should include vertical and horizontal survey measurements accurate to at least 0.01 feet. A baseline reading of the monitoring points should be completed prior to beginning excavation. The survey data should be provided to GeoEngineers for review within 24 hours.

For shoring walls, we recommend that optical survey points be established along the top of the shoring walls. The survey points should be located on every other soldier pile along the wall face for soldier pile and tieback shoring. If lateral wall movements are observed to be in excess of ½ inch between successive readings or if total wall movements exceed 1 inch, construction of the shoring walls should be stopped to determine the cause of the movement and to establish the type and extent of remedial measures required.

APPENDIX H
Report Limitations and Guidelines for Use

APPENDIX H
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of Willow Run, LLC and other project team members for the Building X project. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report Is Based on a Unique Set of Project-specific Factors

This report has been prepared for the Building X project in Redmond, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- Not prepared for you,
- Not prepared for your project,
- Not prepared for the specific site explored, or
- Completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- The function of the proposed structure;
- Elevation, configuration, location, orientation or weight of the proposed structure;
- Composition of the design team; or
- Project ownership.

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org .

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.

GeoEngineers